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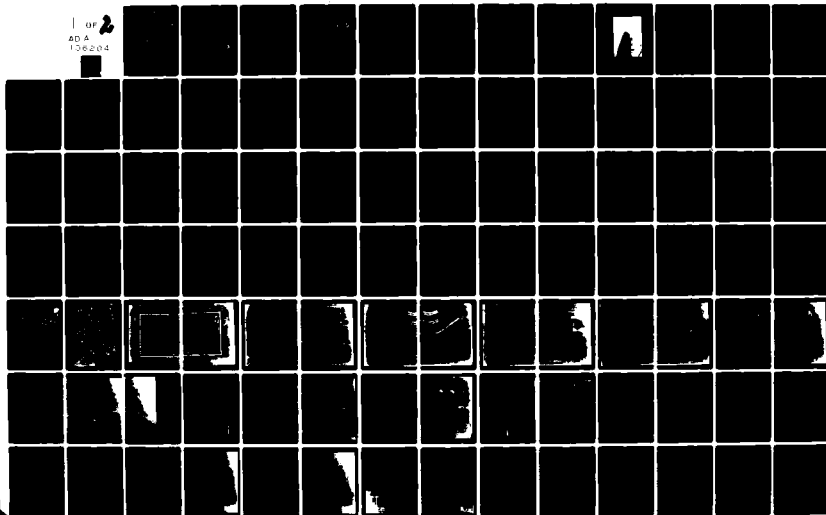
HORNER AND SHIFRIN INC ST LOUIS MO
NATIONAL DAM SAFETY PROGRAM. COMPTON HILL RESERVOIR (MO 31696),--ETC(U)
JUL 81 R E SAUTHOFF, A B BECKER, G K HASEGAWA DACW43-81-C-0002

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UPPER MISSISSIPPI - KASKASKIA - ST. LOUIS BASIN

COMPTON HILL RESERVOIR

ST. LOUIS, MISSOURI

MO 31696

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PHASE 1 INSPECTION REPORT NATIONAL DAM SAFETY PROGRAM



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REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER	2. GOVT ACCESSION NO.	3. RECIPIENT'S CATALOG NUMBER
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4. TITLE (and Subtitle) Phase I Dam Inspection Report National Dam Safety Program Compton Hill Reservoir (MO 31696) St. Louis City, Missouri		5. TYPE OF REPORT & PERIOD COVERED Final Report
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9. PERFORMING ORGANIZATION NAME AND ADDRESS U.S. Army Engineer District, St. Louis Dam Inventory and Inspection Section, LMSED-PD 210 Tucker Blvd., North, St. Louis, Mo. 63101		8. CONTRACT OR GRANT NUMBER(s) DACW43-81-C-0002
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19. KEY WORDS (Continue on reverse side if necessary and identify by block number) Dam Safety, Lake, Dam Inspection, Private Dams		
20. ABSTRACT (Continue on reverse side if necessary and identify by block number) This report was prepared under the National Program of Inspection of Non-Federal Dams. This report assesses the general condition of the dam with respect to safety, based on available data and on visual inspection, to determine if the dam poses hazards to human life or property.		

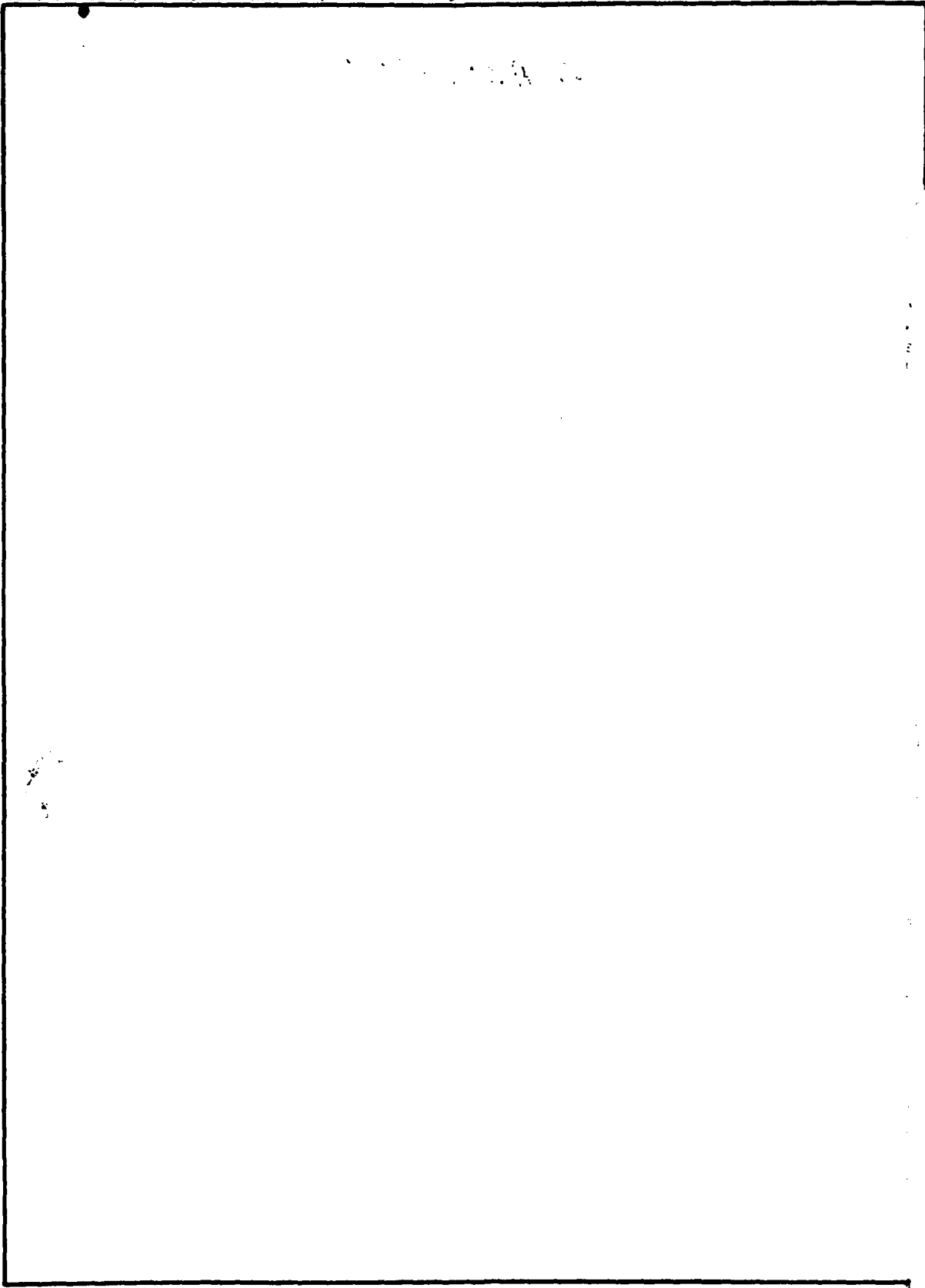
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COMPTON HILL RESERVOIR

ST. LOUIS, MISSOURI

MO 31696

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PHASE 1 INSPECTION REPORT NATIONAL DAM SAFETY PROGRAM



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REPLY TO
ATTENTION 1A

LMSED-P

SUBJECT: Compton Hill Reservoir, MO 31696

This report presents the results of field inspection and evaluation of the Compton Hill Reservoir. It was prepared under the National Program of Inspection of Non-Federal Dams.

SUBMITTED BY: SIGNED 23 JUL 1981
Chief, Engineering Division Date

APPROVED BY: SIGNED 24 JUL 1981
Colonel, CE, District Engineer Date

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COMPTON HILL RESERVOIR
MISSOURI INVENTORY NO. 31696
ST. LOUIS, MISSOURI

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

PREPARED BY:
HORNER & SHIFRIN, INC.
5200 OAKLAND AVENUE
ST. LOUIS, MISSOURI 63110

FOR:
U. S. ARMY ENGINEER DISTRICT, ST. LOUIS
CORPS OF ENGINEERS

JULY 1981

HS-8088

PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

Name:	Compton Hill Reservoir
State Located:	Missouri
City Located:	St. Louis
Stream:	Not Applicable
Date of Inspection:	27 January 1981

The Compton Hill Reservoir has been classified as being of high hazard potential under provision of Public Law 92-367 by the St. Louis District, Corps of Engineers. This report represents an evaluation of potentially hazardous conditions, based upon a study of available engineering data and limited site inspection by engineering personnel of Horner & Shifrin, Inc., Consulting Engineers, under contract to the St. Louis District, Corps of Engineers. Evaluation of this facility was performed in accordance with the "Phase I" investigation procedures prescribed in "Recommended Guidelines for Safety Inspection of Dams", dated May 1975.

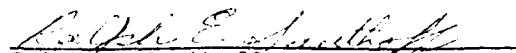
The following summarizes the findings of the visual inspection. Due to the fact that the reservoir is closed (a roof covers both storage basins) and, since the level of the reservoir is not subject to change as a result of storm water runoff, the usual analysis of spillway capacity and overtopping potential was not required. The hydraulic level of the reservoir is controlled by pumping. The level is affected by user demand and by reservoir leakage. Based on the visual inspection and primarily due to the deteriorated condition of portions of the concrete walls about the perimeter of the reservoir, the present general condition of the reservoir is considered to be somewhat less than satisfactory. Although not considered to be an item that could affect the safety of the reservoir, the reported structural condition of the roof support system at the north basin could have, should collapse of a portion of the roof occur, an adverse effect on the future operation of the reservoir.


According to the criteria set forth in the recommended guidelines, the size classification of the reservoir, based on the height of the reservoir relative to the surrounding ground and storage capacity, is intermediate.

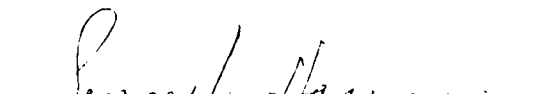
A review of available data did not disclose that seepage and stability analyses of the embankment about the reservoir were performed in accordance with criteria prescribed in the recommended guidelines. This is considered a deficiency and should be rectified. An extensive stability investigation of the north side of the reservoir was made by the Missouri State Highway Department prior to construction of Interstate Highway 44. However, this investigation was performed primarily for the purpose of determining the stability of the retaining wall to be constructed along the south side of the highway adjacent to the reservoir, and did not include the possibility of failure of the reservoir embankment. As a matter of record, it is recommended that the Owner obtain a copy of the Highway Department report describing these investigations and the results of the analyses performed.

The Compton Hill Reservoir is located in a highly populated area of south St. Louis. Failure of the embankment and/or exterior wall of the reservoir could result in loss of life, serious damage to houses, and extensive damage to industrial and commercial facilities, important public utilities, or main highways.

It is recommended that the Owner take the necessary action in the near future to correct the deficiencies and safety defects reported herein.


Ralph E. Sauthoff
P. E. Missouri E-19090


Albert B. Becker, Jr.
P. E. Missouri E-9168


George H. Hasegawa
P. E. Missouri E-4551



OVERVIEW COMPTON HILL, PISCATAWAY

PHASE 1 INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
COMPTON HILL RESERVOIR - MO 31626

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|------|---|

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PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
COMPTON HILL RESERVOIR - MO 31696
SECTION 1 - PROJECT INFORMATION

1.1 GENERAL

a. Authority. The National Dam Inspection Act, Public Law 92-367, dated 8 August 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a program of safety inspection of dams throughout the United States. Pursuant to the above, the St. Louis District, Corps of Engineers, directed that a safety inspection of the Compton Hill Reservoir be made.

b. Purpose of Inspection. The purpose of this visual inspection was to make an assessment of the general condition of the above reservoir with respect to safety and, based upon available data and this inspection, determine if the reservoir poses an inordinate danger to human life or property.

c. Evaluation Criteria. This evaluation was performed in accordance with the "Phase I" investigation procedures as prescribed in "Recommended Guidelines for Safety Inspection of Dams," Appendix D to "Report to the Chief of Engineers on the National Program of Inspection of Non-Federal Dams", dated May 1975.

1.2 DESCRIPTION OF PROJECT

a. Description of Reservoir and Appurtenances. The Compton Hill Reservoir is a rectangular shaped structure approximately 508 feet wide and 836 feet long at the top. The reservoir contains potable water for urban use. Water is contained within an inverted trapezoidal basin by a continuous section consisting of an earthen embankment and a 10-foot high concrete wall, which extends above the crest of the embankment. The height of the embankment varies, but record drawings indicate it to be approximately 26.5 feet above

the toe of slope of the interior side. A center wall separates the reservoir into two equal sections, the north basin and the south basin. A concrete roof supported by concrete beams, which, in turn, are supported by concrete columns, covers the entire reservoir. At normal high water level, the reservoir contains approximately 88 million gallons, which is equivalent to about 270 acre-feet of water. A general plan of the reservoir prepared prior to the construction of the roof, is shown on St. Louis Water Works Drawing 3730 and is included herein, reference Plate 3. An overview photograph of the reservoir taken during the inspection is shown following the preface at the beginning of the report.

A survey conducted as a part of this inspection indicated the earth embankment about the reservoir to vary in height from approximately 21 feet above the original ground elevation at the southwest corner of the structure to about 34 feet at the northeast corner of the structure. However, excavation in 1969 for the construction of Interstate Highway 44 near the north end of the reservoir resulted in an increase in the height of the earthen section at this end to about 40 feet above the roadway pavement. A retaining wall up to 21 feet high extends along Highway I-44 to retain the earth bank along the cut section of the highway. The survey also indicated that outside the reservoir wall, the embankment has a top width of about 18 feet and a slope of about 1v on 2.0h, although the slope steepens to about 1v on 1.9h in some locations. According to record drawings, the slope of the inside face of the embankment is 1v on 1.5h. The interior face is protected by macadam, rock pavement, and two layers of concrete slab with five layers of felt covered asphalt sandwiched between the two slabs. A puddle core of clay constructed along the centerline of the embankment serves as a seepage cutoff. A clay puddle seepage cutoff that joins the clay puddle core of the embankment at about its base, lies beneath the floor and sloping bottom of the structure. A cross-section of the embankment including Highway I-44 is shown on St Louis Water Division Drawings 4536, 4537 and 4545, reference Plates 17, 18 and 19. Cross-sections of the embankment obtained by survey during the inspection are shown on Plates 20, 21, and 22, and the locations of the sections are indicated in plan on Water Division Drawing 3730, reference Plate 3.

According to the construction drawings provided by the Water Division, the concrete wall about the reservoir extends 10.0 feet above the intersection of the original sloping sides of the basin and the inside face of the wall. The top of the wall is also about 9.5 feet above the ground surface, or crest of the embankment, on the outside of the wall, and approximately 38.5 feet above the low point of the bottom of the reservoir. The footing of the outside wall is supported by concrete piles. A continuous steel sheet pile seepage cutoff extends from within the footing to bedrock. The concrete division wall that separates the north and south basins is a gravity section also supported by concrete piles. A continuous concrete wall, or diaphragm, for seepage cutoff extends from the bottom of the wall to bedrock. Details of the exterior walls and division wall are shown on St. Louis Water Works Drawings 3734, 3735, and 3807, reference Plates 5, 6 and 7. Soil borings obtained along the alignment of the exterior walls are shown on Drawing 3731, reference Plate 4.

A gate chamber is located at both ends of the division wall to allow isolation of the north and south basins and to provide some degree of control of inflow and outflow from the reservoir. A remotely controlled, motorized sluice gate is located at each of the openings for the intake and outlet pipes. In all, there are three pipes, two on the east side and one on the west side, that supply the reservoir. Depending on water main pressure, these three pipes can also serve as reservoir outlets along with a fourth pipe that connects to the west side of the structure. Both chambers also contain four manually operated sluice gates, two from each basin, in order to control flow into or out of the basins. The east gate chamber contains an inner chamber, which drains to a 20-inch diameter pipe sewer. This inner chamber is provided to drain the basins, provide an outlet for basin overflow, and to serve as an inlet for stormwater runoff from the roof of the reservoir. The overflow openings at the inner chamber are 1.4 feet high and 6.0 feet wide. Two manually operated sluice gates, one from each basin, control flow from the basins to the inner chamber. St. Louis Water Works Drawing 3730, reference Plate 3, shows a plan of the reservoir including the gate chambers, and Drawing 3807, reference Plate 7, shows details and sections of the two gate chambers as well as a profile of the division wall.

A five inch thick concrete roof supported by beams and columns covers the entire reservoir. At one time, tennis courts occupied a portion of the south half of the reservoir roof. The tennis courts have been removed, but the stanchions for lighting the area still remain. The reservoir roof is normally accessed via a stairway leading to the gatehouse on the west side of the structure. Other stairways located at the corners of the structure and leading to the roof are not normally used. Manholes and hatchways located in the roof serve to provide access to the interior of the reservoir basins. A general plan and sections of the roof are shown on Water Division Drawing 4471, reference Plate 8.

b. Location. Compton Hill Reservoir is located within the City of St. Louis, just southeast of the intersection of Grand Boulevard and Interstate Highway 44, as shown on the Regional Vicinity Map, Plate 1. The reservoir is located within U.S. Survey Number 2498 in Section 21, Township 45 North, Range 7 East. As indicated by the topography shown on Plate 2, the reservoir site is located at about the natural high point of the surrounding area.

c. Size Classification. The size classification based on the height of the reservoir relative to the elevation of the surrounding ground, and storage capacity, is categorized as intermediate. (Per Table 1, Recommended Guidelines for Safety Inspection of Dams.) An intermediate size impoundment is classified as having a height less than 100 feet, but greater than or equal to 40 feet and/or a storage capacity less than 50,000 acre-feet, but greater than or equal to 1,000 acre-feet.

d. Hazard Classification. The Compton Hill Reservoir, according to the St. Louis District, Corps of Engineers, has a high hazard potential, meaning that if the reservoir should fail, there may be loss of life, serious damage to homes, or extensive damage to agricultural, industrial and commercial facilities, important public utilities, main highways, or railroads. Since the reservoir facility is located in a highly populated area of south St. Louis, failure of the embankment and/or exterior wall of the reservoir could result in loss of life, serious damage to homes, and extensive damage to industrial and commercial facilities, important public utilities, or main highways.

e. Ownership. The reservoir is owned by the City of St. Louis. Don C. Guilfooy is the Director of Public Utilities for the City of St. Louis and Raymond F. Walters is Acting Commissioner of the Water Division. Mr. Guilfooy's address is Room 311, City Hall, Twelfth & Market Streets, St. Louis, Missouri 63103. Mr. Walter's address is 1640 South Kingshighway, St. Louis, Missouri 63110. Stanley T. Fletcher, Division Engineer, Design and Construction Section, Joseph J. Kammerer, Jr., Assistant Division Engineer, Design and Construction Section, and Thomas A. Rothermich, Division Engineer, Operating Section, all of the City of St. Louis Water Division, served as the Owner's representatives during the course of the investigations reported herein.

f. Purpose of Reservoir. The reservoir impounds potable water to satisfy peak demands and provide capacity for fire fighting.

g. Design and Construction History. According to a brass plaque that had been placed at the reservoir site and now lies in storage at the Kingshighway Avenue Office of the Water Division, the original embankment and slope protection was constructed from 1868 to 1870 by the Murphy-Henderson Company, a St. Louis Contractor. The plaque also indicated that Thomas J. Whitman was Chief Engineer for the project and James P. Kirkwood was the Consulting Engineer. Construction plans were available at the Water Division's office, but due to the rather fragile condition of these plans, they were not removed from their file drawers. They were, however, reviewed by the inspection team. Engineering design data relating to this phase of construction of the reservoir were unavailable.

Another plaque in storage indicated that the reservoir was reconstructed during 1915 and 1916 by the Hiram Lloyd Building Construction Company. According to the plaque, Gordon G. Black was the Engineer in charge of the project, and a firm by the name of Roth & Sturdy were the Architects. According to Mr. Kammerer and plans on file with the Water Division, the exterior concrete walls, the interior division wall, and the concrete slabs and waterproofing on the interior slopes and bottom of the reservoir were constructed at this time. Detailed engineering computations for design of the walls were available at the Kingshighway Avenue Office of the Water Division.

Several drawings relating to the reconstruction of the reservoir, reference Plates 3 through 7, are included herein.

Mr. Kammerer reported that the concrete roof covering the reservoir was constructed in 1933. With the exception of several drawings showing details of the roof and supports prepared by the Water Division, information regarding the design of the roof was not available. A drawing showing the roof and its supporting members, reference Plate 8, is included herein.

h. Normal Operational Procedure. The level of the reservoir fluctuates depending upon main pressure and user demand. The reservoir is supplied with potable water by pumping from the City's Chain of Rocks Treatment Plant through what is referred to as the Compton Hill Reservoir System. However, it can also be supplied through a number of bleeder valves that connect to the City's Stacy Park Reservoir System. The Compton Hill System is termed a low service system, whereas the Stacy Park System is a high service system. The Chain of Rocks Plant is located on the west side of the Mississippi River at about river mile 190, which is approximately 10 miles north and 3 miles east of the reservoir. The Stacy Park Reservoir, which stores potable water supplied by the Howard Bend Treatment Plant located on the Missouri River, is located approximately 9 miles west and 4 miles north of the reservoir. Pumping rates to the reservoir are increased at night and on weekends to take advantage of off-peak electrical costs. Historically, the water level within the reservoir is at its maximum elevation following weekends, lowers during the day, rises at night, and is at its lowest elevation just prior to the weekend. Unusual water demands such as that caused by fire fighting may cause deviation from the normal operating procedures. The relative locations of the Howard Bend Plant, the Stacy Park Reservoir, the Chain of Rocks Plant, and the Compton Hill Reservoir, along with water mains larger than 12 inches, are shown on Water Division Drawing Exhibit A, reference Plate 9.

1.3 PERTINENT DATA

a. Drainage Area. The reservoir has no inflow due to precipitation and resulting runoff. The reservoir is covered by a roof, and roof drainage is sewered.

b. Discharge at Reservoir.

- (1) Reservoir inflow/outflow ... Unknown
- (2) Drain capacity ... 30 cfs (20-inch pipe sewer)

c. Elevation. Except where noted, the following elevations were determined from record drawings and/or survey. Elevations obtained by survey are based on a bench mark established at the reservoir by the City. The bench mark, a chiseled square in concrete reported to be elevation 190.05, City datum, is located on the top step of the doorway leading to the roof of the reservoir of the west gate house. The conversion from the City of St. Louis datum to USGS (ft. above sea level) datum is City datum +413.536. Elevations shown are in City datum followed by the USGS datum in parenthesis. Topography in the vicinity of the reservoir, obtained from the 1954 USGS Cahokia, Illinois-Missouri Quadrangle Map (photorevised 1968 and 1974) is shown on Plate 2. The topography shown on Plate 2 is in USGS Datum.

- (1) Observed level ... 182.5 (596.0)
- (2) Normal level ... Varies
- (3) Normal high water level ... 185.4* (598.9)
- (4) Overflow crest ... 186.6* (600.1)
- (5) Overflow alarm level ... 186.4* (599.9)
- (6) Top of grated opening at sewer chamber ... 188.25 (601.8)
- (7) Maximum experienced level ... Unknown
- (8) Top of reservoir wall ... 190.0 (603.5)
- (9) Top of roof at wall ... 190.4 (603.9)
- (10) Top of division wall ... 188.0 (601.5)
- (11) Basin floor at toe of slope ... 154.0 (567.5)
- (12) Basin floor (min.) ... 151.5 (565.0)
- (13) Bottom of sewer chamber ... 151.1 (564.6)
- (14) Invert 20-inch sewer ... 150.2 (563.7)
- (15) Streambed at centerline of dam ... Not applicable
- (16) Maximum tailwater ... Not applicable

*Per Mr. Thomas A. Rothermich, Division Engineer, Operating Section, City of St. Louis Water Division.

d. Impoundment (North and South Basins).

- (1) Length at maximum level ... 831.5 ft. (interior, per Plate 3)
- (2) Width at maximum level ... 502.5 ft. (interior, per Plate 3)

e. Storage Volume (Total North and South Basins).

- (1) Normal high level (El. 185.4) ... 270 ac.ft. (88.0 MG)
- (2) Overflow level (El. 186.6) ... 281 ac.ft. (91.5 MG)

f. Reservoir Surface Area.

- (1) Normal high level (El. 185.4) ... 9.6 acres
- (2) Overflow level (El. 186.6) ... 9.6 acres

g. Reservoir. The height of the reservoir is defined to be the overall vertical distance from the lowest point of foundation surface to the top of the structure.

- (1) Type ... Combined section: concrete wall (upper 10.0 ft.)
above earthfill embankment
- (2) Basins
 - a. Number ... Two (equal size)
 - b. Width (each) ... 414.25 feet (inside at top)
 - c. Length (each) ... 502.50 feet (inside at top)
- (3) Embankment height ... 40 ft. (max. at I-44)
- (4) Height of total section ... 50 ft. (max. at I-44)
- (5) Top width of embankment ... 18 ft. (outside of wall)
- (6) Side slopes of embankment
 - a. Interior ... 1v on 1.5h*
 - b. Exterior ... 1v on 1.9h (max.)

*Per 1868-1870 construction drawings.

- (7) Seepage cutoff
 - a. Clay puddle*
 - b. Steel sheet piling**
- (8) Embankment slope protection
 - a. Interior ... Two concrete slabs with a waterproof membrane over rock pavement and macadam**
 - b. Exterior ... Grass
- (9) Inflow/Outflow pipes ... 1-48", 2-36", 1-30" (all controlled)
- (10) Overflow ... Two 1.4 ft. high by 6.0 ft. long openings, one each basin (uncontrolled)

h. Overflow (Typical Each Basin).

- (1) Type ... Uncontrolled
- (2) Size ... 1.4 feet wide by 6.0 feet long
- (3) Location ... East gate chamber at sewer well
- (4) Invert ... Elevation 186.6 (600.1)

i. Reservoir Drain Facility.**

- (1) Control ... Two 24-inch wide by 30-inch high sluice gates
- (2) Gate invert elevation ... 151.6 (565.1)
- (3) Outlet ... 20-inch diameter pipe sewer
- (4) Location ... East gate chamber
- (5) Sewer invert elevation ... 150.2_± (563.7)
- (6) Estimated time required to drain basin at normal high level
 - a. At 30 cfs ... 55 hours (max., discharge without flooding)
 - b. At 50 cfs ... 33 hours (with flooding at manhole)

*Per 1868-1870 construction drawings.

**Per 1915-1916 construction drawings.

SECTION 2 - ENGINEERING DATA

2.1 DESIGN

a. As previously indicated, data relative to the design of the reservoir as originally constructed in 1868-1870 were limited to a group of drawings of the reservoir on file at the Kingshighway Avenue office of the Water Division. These plans were reviewed by the inspection team, but due to the fragile condition of these drawings, were not removed from the Water Division office and are not included herein. According to details shown on these plans, the slope of the inside face of the embankment is 1v on 1.5h, the original width of the embankment crest was 20 feet, and the slope of the exterior face of the embankment is 1v on 2.0h. Also shown on the drawings was the original division wall separating the north and south basins. With the exception of the information shown on these plans, no other design data were available. Records did indicate that Thomas J. Whitman was Chief Engineer for the project and James P. Kirkwood was project Consulting Engineer.

b. In 1915-1916, certain improvements were made by the City to the reservoir. Records indicated that a firm by the name of Roth & Sturdy were the Architects for this reconstruction project and that the Engineer in Charge was an individual by the name of Gordon G. Black. These improvements consisted of a two course reinforced concrete pavement slab to be placed above the original floor of the bottom and sloping sides, a reinforced concrete division wall to replace the existing division wall, two reinforced concrete gate chambers, one at each end of the new east-west division wall, and a reinforced concrete wall to encompass the reservoir. Details of these features are shown on Water Division Drawings 3730, 3734, 3735, and 3807, reference Plates 3, and 5 through 7. Test borings to rock along the line of the outside wall were obtained by the City and the results of these investigations are shown on Water Division Drawing 3731, reference Plate 4. As indicated by the details shown on the drawings, all of the structures are supported by reinforced concrete piles and both the division wall and the gate chambers have a 3-foot wide continuous concrete wall that extends from the bottom of the structure to rock. This wall acts as both a support for the structure and a seepage cutoff diaphragm. Details on the drawings indicated

the wall about the perimeter of the reservoir has a continuous steel sheet piling seepage cutoff diaphragm that extends from the base of the structure to rock. From information shown on the drawings, it can be seen that the new pavement slabs and division wall were provided with expansion joints; however, no such details are given for the side walls and none were noticed during the inspection. Engineering computations for the design, including stability analyses, of these walls were on file at the Kingshighway Avenue office of the Water Division. A review of these computations indicated the design of the walls to be satisfactory.

c. The reservoir was again improved in 1933 by the addition of a roof to cover the north and south basins. Details of the roof system, including the locations of columns supporting the roof beams, are shown on Water Division Drawing 4471, reference Plate 8. As indicated on the drawing, the bases for support of the columns on the sloping portion of the basin floor are continuous, extending from the vertical wall at the top of the slope to the near level area of the floor one bay in from the toe of the slope. Additional drawings of the roof system showing details of the beams and columns are on file at the office of the Water Division. However, design computations for these members could not be located.

d. In about 1967, a detailed engineering investigation of the north side of the reservoir was made by the Missouri State Highway Department for the purpose of determining the stability of the reservoir embankment under conditions to be imposed by Interstate Highway 44. As previously indicated, Highway I-44 lies parallel to and just north of the reservoir. The highway in this area is a depressed section with a retaining wall, that varies in height from about 21 feet at the west end of the reservoir to approximately 10 feet at the east end of the facility, located adjacent to the south side of the highway. The retaining wall is a reinforced concrete, counterfort type, section with its footing bearing on and keyed into rock. The wall backfill is primarily sand and a continuous 18-inch diameter pipe drain with a crushed stone drainage blanket, that covers the footing and the drain pipe, is located at about the heel of the footing. The location of the retaining wall relative to the north side of the reservoir is shown on Water Division Drawings Numbers 4532, 4533, 4536, and 4545, reference Plates 15, 16, 17, and 19, respectively.

Cross-sections of the north embankment, including the highway retaining wall, based on survey data obtained during the inspection, are shown on Plate 20. A report, dated April 4, 1967, describing the investigations made by the Highway Department during the course of their study of conditions affecting the proposed retaining wall, as well as recommendations for construction of the wall, was prepared by Mr. Roy Rucker, Materials and Research Engineer, Missouri State Highway Department. Highway Department policy did not permit release of their report at this time for inclusion within this report, although it is understood that it may be made available upon request by the City. The report, however, was reviewed by the inspection team.

The report indicates that two lines of piezometers were installed within the north slope of the reservoir for the purpose of obtaining ground water levels. It also states that several test borings were made in the vicinity of the slope, and that undisturbed samples of the soil encountered in these borings were obtained. Vane shear tests were also made of softer and wetter zones of material encountered in the borings. During the course of these subsurface investigations, the report states that the most significant finding was the unexpectedly high water elevations within the piezometers and the lower rock line, both in the vicinity of the northeast corner of the reservoir. Chemical analysis of the water indicated it to be leakage from the reservoir. The soils encountered were identified as being of loessal origin overlying residual clay. In addition to vane shear tests, direct shear tests of saturated soil samples and unconfined compression tests were performed. The most consistent and reliable data were considered to be that obtained from the direct shear tests, and this data was used in the stability analyses. Strength values for the loess and residual clay were adopted from these tests for use in stability analyses.

According to the report, stability analyses were performed using both the block and wedge method and the Swedish circle method, although the block and wedge method was considered the more appropriate of the two and was used during final design. Certain active wedge and hydrostatic pressure conditions were assumed and analyses were made. For the construction condition, the report indicates a minimum factor of safety of 0.9 with the water level (of the reservoir) at elevation 588 (elevation 174.5 City datum) and 1.25 with the

water level at elevation 567 (elevation 143.5 City datum). A minimum safety factor of 1.45 is given for final (after construction) condition.

The report also addresses the problem of reservoir settlement due to dewatering of the basin during construction of the wall adjacent to the highway. A statement in the report indicates that it cannot be positively stated that no adverse effects to the reservoir from the proposed cut, backfill and drain method of construction will occur. This is believed to be in reference to the type of wall shown on the highway exhibits included in the report. The exhibits show a reinforced concrete or crib wall founded on earth above bedrock. An alternate design consisting of a buttressed wall of reinforced concrete keyed into bedrock is recommended. Additional recommendations regarding construction methods and dewatering of the north basin of the reservoir, are also presented.

Other correspondence in the Highway Department file indicated the buttress (counterfort) wall should be designed for lateral earth pressures of 70, 85 and 100 psf depending upon the location of the section, that the wall should be designed for possible development of hydrostatic pressure from full reservoir level down to bedrock, and that the wall footing should be embedded a minimum of 6 inches into sound rock in addition to being keyed into rock at least 12 inches to prevent sliding. Another letter in the file recommends replacement of excavated earth (backfill) with sand backfill, the use of crushed rock about the subdrain pipe, and the installation of several permanent piezometers in the north slope for the purpose of monitoring ground water levels between the reservoir and the highway retaining wall.

From the details shown on Plate 17 and Plate 19 as well as observations made in the field during the inspection, it would appear that these recommendations were closely followed.

2.2 CONSTRUCTION

As previously stated, the reservoir was originally constructed between 1868 and 1870 by a general contractor by the name of Murphy-Henderson. With the exception of the construction plans on file with the Water Division, very

little is known regarding this construction. If records of these activities exist, they are somewhere within the archives of the City.

As reported elsewhere in this report, the Hiram Lloyd Building Construction Company was the contractor responsible for the reservoir reconstruction work that occurred between 1915 and 1916. Copies of a number of the construction drawings prepared for reconstruction of the reservoir are included herewith, reference Plates 4 through 7. Additional drawings of the required reconstruction are on file at the Kingshighway Avenue office of the Water Division. No other records of these construction activities were available.

Also, as previously reported, the reservoir basins were covered with a reinforced concrete roof system in 1933. A drawing, reference Plate 8, showing a general plan of the basin cover along with sections through the basin that show the roof slab and means of support is included herein. The name of the contractor who constructed the roof is unknown, and information relating to the actual construction was unavailable.

2.3 OPERATION

The level of the reservoir is governed by system pressure, which fluctuates with user demand. The reservoir is normally supplied by pumping potable water from the Chain of Rocks Water Treatment Plant. However, the reservoir can also be supplied from the Stacy Park Reservoir high service system through a number of remotely controlled bleeder valves. The valve on the 48-inch diameter steel pipe located near the intersection of Compton Hill Place (formerly Louisiana Avenue) and Russell Avenue is the valve most commonly used for this purpose. A 36-inch diameter pipe that connects to the south side of the east gate chamber delivers the Stacy Park System water to the reservoir. Flow from the Chain of Rocks Plant enters the reservoir through a 48-inch diameter pipe that connects to the north side of the east gate chamber and through a 30-inch diameter pipe that connects to the north side of the west gate chamber. Reservoir outflow can occur at four locations. In addition to the two influent lines from the Chain of Rocks Plant and the line that connects to the Stacy Park System, which can also serve as effluent

lines should system pressure dictate, a 6-inch diameter pipe exits the reservoir at the south side of the west gate chamber. Remotely controlled motorized sluice gates located within the gate chambers are provided to prevent reservoir outflow in the case of a break in these lines. Water Division Drawings 36 and 36A showing the reservoir and the water main distribution system in the vicinity of the reservoir are included herein, reference Plates 11 and 12. Distribution main symbols are shown on Water Division Drawing A-197, reference Plate 10. The locations of the Stacy Park Reservoir and the Chain of Rocks Plant, including mains larger than 12 inches, are shown on Water Division Drawing Exhibit 3, reference Plate 9.

A concrete wall divides the reservoir into two equal basins, the north basin and the south basin. Four manually operated sluice gates within the east and west gate chambers allow flow to reach both basins, or allow the basins to be operated independently. The normal method of operation is to supply both basins simultaneously and to operate the basins in parallel.

A sensing device attached to a bubbler tube located in the west gate chamber transmits via telephone lines the level of the reservoir to the pump control room at the Chain of Rocks Plant and to the Kingshighway Avenue office of the Water Division. An alarm is signalled at the Chain of Rocks Plant and at the Kingshighway Avenue office in the event of excessive reservoir level. The alarm level is set 1.0 foot above the normal high water level. A staff gage located within the west gate house also indicates the level of the reservoir and is used to check the level transmitted by the telemetering system. A gasoline operated generator located within the west gate house is available to provide electrical power to the telemetering system in the case of an electrical outage. Two rectangular openings, each approximately 1.4 feet high and 6.0 feet wide, located in the gatewell at the east gate chamber, are provided for basin overflow. The inverts of the overflow openings are about 0.2 of a foot above the high water alarm level. Two manually operated sluice gates located in the gatewell at the west end of the east gate chamber are provided to drain the basins. A 20-inch diameter sewer located in the bottom of the gatewell serves as an outlet for reservoir drainage. The east and west gate chambers including the locations of the gates within the chambers and the sewer pipe at the east gate chamber, are shown on Water Division Drawing 3730, reference Plate 3.

2.4 EVALUATION

a. Availability. Engineering data for assessing the design of the reservoir were available. Readily available data was limited to the details shown on the original, 1868-1870, plans for construction of the facility, the 1915 set of plans for reconstruction of the reservoir, the computations for the design of the walls, etc., constructed in 1915-1916, and the drawings prepared in 1931 for construction of the roof system.

Seepage and stability analyses of the embankments of the reservoir comparable to the analyses prescribed by the recommended guidelines were not available.

b. Adequacy. Considering the long history of satisfactory performance, the inspection team is of the opinion that the details shown on the construction plans for the reservoir are adequate to assess the design of the facility, which is considered, in general, to be satisfactory. However, as indicated in Section 4, paragraph 4.5, certain improvements to the reservoir are necessary to insure the continued satisfactory performance of the facility. As indicated above, seepage and stability analyses of the embankments of the reservoir comparable to the requirements of the "Recommended Guidelines for Safety Inspection of Dams" were not available. These seepage and stability analyses should be performed for appropriate loading conditions (including earthquake loads) and made a matter of record.

SECTION 3 - VISUAL INSPECTION

3.1 FINDINGS

a. General. A visual inspection of the Compton Hill Reservoir was made by Horner & Shifrin engineering personnel, R. E. Southoff and G. K. Hasegawa, Civil Engineers, and A. B. Becker, Jr., Civil and Soils Engineer, on 27 January 1981. Messrs. Kammerer and Fletcher of the City of St. Louis Water Division were present during this inspection. An examination of the reservoir area was also made by an engineering geologist, Jerry D. Higgins, Ph.D., a consultant retained by Horner & Shifrin for the purpose of assessing the site geology. Also examined at the time of the inspection were the areas and features below the reservoir within the potential flood damage zone. Photographs of the reservoir taken at the time of the inspection are included on pages A-1 through A-7 of Appendix A. The locations of the photographs taken during the inspection are indicated on Plate 3.

b. Site Geology. The Compton Hill Reservoir is located in the southern portion of the City of St. Louis. Most of the original topography and drainage in this area have been modified by urban development; however, the older topographic maps indicate that the original surface drainage was away from Compton Hill into small creeks that drained into the Mississippi River. Some of the drainage undoubtedly was internal into the river through sinks developed in the soluble bedrock. The site is located on the northeastern flank of the Ozark Plateaus Physiographic Province.

In the general area of the reservoir, the bedrock consists of Mississippian- and Pennsylvanian-age sedimentary strata which dip gradually toward the northeast into the Illinois Basin. There are no bedrock exposures at the reservoir; however, there are several outcrops in the I-44 roadway cut just north of the site. The reservoir is located on the Mississippian-age St. Louis formation. The bedrock is a gray, finely crystalline, medium- to massive-bedded limestone. The formation has been subjected to extensive solution weathering, and caves, sinks, and other karst topographic features are common. Numerous sinks, now filled as a result of city development, were located along a northeast-southwest trend several blocks east of the reservoir

(Missouri Geological Survey file map). Possible geologic problems associated with these features include reservoir leakage, and although unlikely, sink collapse.

The soils at the site have been disturbed and reworked by construction activity, but originally consisted of loessal soils (ML, Unified Soil Classification System) over the red residual clays (ML-CL) derived from weathering of the limestone. Most of the area is now paved or extensively modified, and only the residual soils immediately above the limestone bedrock in the I-44 highway cut may be considered to be undisturbed.

The most significant geologic conditions noted at the site are the karst features associated with the limestone bedrock. The presence of numerous sinks and caves suggest extensive solution weathering of the bedrock. The reservoir has been functioning for many years, and there is no suggestion, from the geologic reconnaissance, that any geological conditions have been detrimental to its performance. However, the karst features are prominent in the area and potentially could adversely affect the reservoir.

c. Reservoir. The exterior portions of the earthen embankment, reinforced concrete side walls, and roof of the reservoir were examined and, except as noted herein, were found to be in sound condition. (The interior of the reservoir was not examined at the time of the inspection due to the presence of water within the reservoir.) Photographs of the reservoir walls and embankment slopes taken during the inspection are shown on pages A-1 through A-4 of Appendix A, and cross-sections of the embankment obtained by field survey at the time of the inspection are shown on Plates 20, 21 and 22. No cracking of the surface, undue settlement of the crest, or significant erosion of the embankment was noted. Examination of soil samples obtained from the four exterior sides of the reservoir embankment at about the center of each side indicated the surficial material of the embankment to be a medium-to-dark brown silty lean clay (CL) of low-to-medium plasticity.

A small bulge (see Photo 2) that extended westwardly approximately 80 feet from the stairway at the northeast corner of the reservoir was noticed in the face of the north embankment at a point just above the toe of slope at the

location of a reported (1972) slope failure. However, no evidence of seepage was visible on the surface of the slope, and no signs of recent movement or other indications of instability of the slope were noted, although the sidewalk which parallels the north slope near the toe appeared to have settled approximately 2 inches relative to the concrete stairway and landing located at the northeast corner of the embankment. The crest of the embankment was uniform and with the exception of an 8-inch diameter tree stump with sprouts located adjacent to the wall near its midpoint, was uniformly covered with grass. Although flow was not evident, water was visible within the two manholes for the 4-inch underdrain near the top of the embankment. As indicated in Section 4, paragraph 4.2f, the subdrain flows were measured on February 6, 1981, and the data obtained are indicated on Chart 4-47. Five of the original seven piezometer tubes (see Photo 3) that had been installed in 1977 were located and found to be in serviceable condition. Ground water levels were measured on February 6, 1981 at the five piezometers located, and the elevations of the water surface are indicated in the schedule on Chart 4-48. Observed groundwater levels were not abnormally high, and in general, were found to agree with the data shown by the City on Plate 19. Piezometer No. 4 is shown in Photo 3.

The exterior portions of the east and south embankments of the reservoir (see Photos 5 and 8) were inspected and appeared to be in satisfactory condition. Standing water was present in a low area near the toe of the east embankment at the north end adjacent to the driveway accessing a storage building. The origin of the water could not be determined, but its location indicated the water was, in all likelihood, a result of stormwater runoff.

The exterior face of the west embankment (see Photo 11) was found to be somewhat irregular along the northern portion due to numerous ruts caused, reportedly, by the tractor used to cut the grass on the slope. The turf cover was sparse within the ruts. An 8-inch diameter vitrified clay pipe sewer had been excavated along the south side of the stairway which ascends the west embankment at its center. The sewer, which was under repair, was leaking at a rate estimated to be less than 1 gallon per minute through a 1-inch diameter hole in the pipe. The source of the flow in the sewer could not be determined.

The exterior side of the concrete wall which encompasses the reservoir at the top of the embankment, was examined and found, at numerous locations, to be extensively deteriorated as a result of spalling of the surface. Numerous hairline type cracks were noted at these spalled areas; however, no major structural defects were observed. Expansion or contraction joints were not visible in the wall, and most of the cracks observed tended in a vertical direction, although a good number of horizontal cracks were also noticed. Due to the fact that the wall had been surface coated (in 1960) by guniting, it is possible that not all of the cracks in the wall were seen. Lime deposits (see Photo 20) were noticeable at many of the cracks indicating some seepage of water through the wall. The exterior face of the north wall (see Photo 1) appeared to be in good condition with only a few relatively small areas of surface spalling. The surface of the east wall (see Photo 4) was marked with several large areas of spalling. Most of the spalls were shallow; however, at one location just north of the center of the east wall, the deterioration at the top of the wall extended approximately 15 inches (see Photo 6) beneath the roof slab. The upper 2 feet of the south wall (see Photo 7) was found to be spalled across much of the length of the wall. At the top of the south wall, the spalling extended up to six inches beneath the roof slab (see Photo 9) and the reinforcing bars were exposed at several locations. The west wall (see Photo 10) also had several large areas of spalling (see Photo 12) and again, the reinforcing bars (see Photo 21) were exposed at several locations. In many instances where the depth of the spall was quite small, say on the order of one-quarter of an inch, the spalled section included the gunited surface coating and a thin layer of the original wall surface.

The overall width and length of the reservoir structure at the level of the top of the embankment was measured during the inspection for the purpose of verifying the dimensions show on the plans. According to the dimensions shown on Water Division Drawing 3730, reference Plate 3, the overall width of the structure is 505.5 feet and the overall length is 834.5 feet. Surveyed distances indicated the overall length of the north side to be 507.3 feet, the south side to be 507.8 feet, the east side to be 836.4 feet, and the west side to be 835.6 feet. The survey also indicated the outside face of the reservoir wall, which is shown on Water Division Drawing 3735, reference Plate 6, to be a vertical wall, to be out of plumb with the plane of the wall leaning inward,

or toward the center of the structure, at each of the four, one at each side, locations checked. At a distance approximately 6.0 feet above the ground, the deflection varied from 0.12 feet for the north wall to 0.21 feet for the south wall, and from 0.19 feet for the east wall to 0.24 feet for the west wall. (It is possible that the wall was given a slight inward batter during construction in order to give the section a more stable appearance. In any event, due to the rather uniform tilt of the wall and the fact that deflection of the wall due to load would be in the opposite direction than that observed, since it is a cantilever, the inspection team is of the opinion that there is no significant foundation or structural problem associated with this deflection.)

The top side of the concrete roof was examined and, except as noted herein, appeared to be in satisfactory condition. The northern one-half of the roof, which had been resurfaced (in about 1948) with a sand-cement grout, had only a few minor hairline type cracks and appeared to be in very good condition, although stains in some areas indicated that minor ponding may occur on the roof as a result of poor drainage following periods of precipitation. The southern one-half of the roof was surfaced with what appeared to be a tar and gravel seal coat. The seal coat was lacking at many locations and several areas of spalling of the concrete surface of the roof were evident. Additional deteriorated material (type unknown) was present on the roof (see Photo 13) within the areas which at one time served as tennis courts. Steel columns which are believed to have supported lights for the tennis courts (the courts are no longer used and the general public is not permitted on the reservoir roof) were located adjacent to the tennis courts. As previously indicated, the roof support columns and beams and the underside of the roof was not inspected due to the presence of water within the reservoir.

Both the interior and exterior of the west gate house (see Photo 14) as well as the exterior of the east gate house (see Photo 15) were inspected and found to be in satisfactory condition. The equipment within the west gate house and the operators at the east gate chamber were also examined but not operated as a part of the inspection. The motorized gate operators (see Photo 16) for the effluent pipes on the west side of the reservoir, and the

telemetering control panel (see Photo 18) within the west gate house appeared to be relatively new and well maintained. The staff gage, manual gate operators (see Photo 17) as well as the standby generator (see Photo 19) within the west gate house and the manual operators at the east gate chamber (see Photo 15) were examined and also found to be well maintained and in satisfactory condition.

d. Appurtenant Structures. No appurtenant structures were observed at the reservoir.

e. Downstream Channel. As indicated on Plate 2, the Compton Hill Reservoir is located upon the crest of a hill. Interstate Highway 44 lies parallel and adjacent to the north side of the reservoir site. A City park surrounds the reservoir, but is more pronounced on the east and west sides than on the north and south sides of the facility. Residential areas adjoin the reservoir park on the east, west and south sides, and a hospital borders Grand Avenue at the west side of the facility.

f. Reservoir. At the time of the inspection, according to the staff gage in the west gate house, the water level within the reservoir was at elevation 182.5, City datum, which is about 2.9 feet lower than the normal high water operating level and approximately 4.1 feet below the level of the overflow openings in the east gate chamber. According to a chart within the west gate house of the reservoir, this level corresponds to storage equal to 78.6 million gallons which is equivalent to about 241.2 acre-feet of water.

3.2 EVALUATION

The deficiencies observed during the inspection and noted herein are not considered significant to warrant immediate remedial action. However, it is recommended that, as soon as practical, provisions are made to prevent further deterioration of the concrete walls about the reservoir.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 PROCEDURES

The water level within the reservoir is governed by system pressure, which fluctuates with user demand. The reservoir is supplied with potable water by pumping from the City operated Chain of Rocks Treatment Plant. However, it can also be supplied, through a number of cross connections with bleeder valves, from the City's Stacy Park Reservoir System. Flow from the treatment plant is increased at night and on weekends to take advantage of lower electrical costs. The level of the reservoir is also affected by leakage. Additional information relative to the operation of the reservoir is presented in Section 2, paragraph 2.3.

4.2 MAINTENANCE OF RESERVOIR

a. Routine maintenance of the exterior slope of the embankment, such as mowing the grass, is performed by the St. Louis Parks Department. All other reservoir maintenance is performed by the Water Division. The exterior of the reservoir, i.e., the embankment slopes, retaining walls, roof, etc. is routinely inspected by Water Division personnel. A summary of items pertaining to maintenance for which records are available follows:

b. Leakage tests were performed by the St. Louis Water Department during the period from 1900 to 1902. Records of the tests were available, but have not been included herewith since the reservoir was raised approximately 10 feet and the lining of the bottom and interior embankment slopes has been significantly modified since that time. A report dated January 23, 1901, indicated the leakage from the north basin to be 0.62 MGD (82,300 cubic feet per day), and leakage from the south basin to be 1.94 MGD (258,200 cubic feet per day).

c. *An Annual Report of the Water Commissioner of the City of St. Louis for the Fiscal Year ending April, 1903*, reference Chart 4-1, records the repairs made on the stone pavement of the south basin during the previous summer and fall. It also states that leakage tests of this basin were made

before and after grouting the pavement, and that the results of these tests indicated a decrease from 156,000 cubic feet to 118,300 cubic feet in a 24-hour period. In addition, the Report states that several cavities approximately 10 feet in diameter and 7 or 8 feet deep appeared in the slope of the reservoir near the middle of the east side during January of 1903. According to the Report, the depressions, which may have been caused by sink holes, were filled with earth and stone.

d. On April 19, 1973, an inspection of the north basin of the reservoir was made by personnel of Sverdrup & Parcel and Associates, Engineers-Architects, St. Louis, Missouri, along with representatives of the City Water Department. According to a report prepared by Sverdrup & Parcel, reference Charts 4-2 through 4-6, the purpose of this inspection was to determine the probable cause of excessive basin leakage and to recommend procedures for sealing the basin, at least to the point where the leakage from the basin is acceptable.

The report states that the north basin of the reservoir was drained during construction of Highway I-44, and that after the highway was completed, it was refilled. However, soon afterwards, an objectionable amount of leakage was noticed, and subsequently the basin was drained again for inspection and repair. In addition to the problem of excessive leakage, the report states that there are many instances of structural problems in connection with the roof system and its supports. However, due to the demands for water during the upcoming summer season, repairs to the roof system would have to be postponed until the end of summer.

The report argues that the most apparent cause of the leakage was the drainage of the basin for construction of the highway, and reasons are given to support this hypothesis. It also states that the structural problems associated with the roof are due mainly to the fact that, except at the perimeter of the roof, there are no provisions for expansion and contraction of the roof slab. The writer mentions that movement of the roof, a section about 400 feet by 500 feet, has produced stresses in the roof columns resulting in tension cracks and compressive crushing of the concrete, as well as many instances of cracking and spalling of beams immediately adjacent to

the columns. It is also pointed out that the expansion joints between the perimeter columns and the roof slab beams are no longer functioning effectively in many locations, and that the columns in many places have been pushed aside until the beam bearing area is reduced to as little as one inch. Other, but not as serious, structural problems associated with the roof system are indicated.

The report gives recommendations for repairing the basin floor slab expansion joints. It is also recommended that the repair work be concentrated in the areas of the sloping slabs on the north side of the basin as well as along the north ends of the east and west sides and in the first bay of the flat portion of the floor immediately adjacent to the sloping slabs. A recommendation is also made that the outside of the wall footing in the area of the greatest leakage be excavated in order to inspect the bottom of the wall footing. The report states that if voids exist beneath the footing, the spaces should be sealed with grout.

The author continues by indicating that postponing the structural repairs to the roof system until after the summer season is an acceptable risk, since a failure of the roof would be partial only and would not involve the entire roof, or the basin walls. The report states that following the summer demand, when the basin is drained to allow further repair work, that the level of the basin be reduced slowly, at the rate of one to two feet per day, in order to allow hydrostatic pressure on the underside of the basin floor to dissipate and not force out the expansion joint material or uplift the slab.

In conclusion, the report states that a comprehensive engineering investigation of the north basin is required to determine the required structural repairs as well as the most feasible and economical means of accomplishing the repairs. It recommends that the south basin also be inspected and repaired where necessary in a manner similar to that used in the north basin. The report states that with a few major improvements, the reservoir structure should continue to function satisfactorily for quite some time. However, it does caution that replacement of the expansion bearing plates and the rehabilitation and stabilizing of the columns supporting the perimeter beams be given the highest priorities.

e. Leakage tests of the reservoir basins were performed by personnel of the Water Division during February and March of 1973 and a report entitled "Findings at Compton Hill Reservoir, Spring 1973", was subsequently prepared. The report, which is dated August 9, 1979 also describes the structural condition of the two basins as found during a recent inspection and a series of excavations made to check the condition of the underside of the footing for the exterior wall at the north side of the reservoir. This report is included herewith, reference Charts 4-7 through 4-32. According to the author, Mr. Joseph J. Kammerer, Jr., Assistant Division Engineer, Design and Construction Section, City of St. Louis Water Division, the inspection of the basins was made at about the same time as the Sverdrup & Parcel inspection, and the excavations of the footings for the north wall were made as a result of a recommendation contained in the Sverdrup & Parcel report.

According to the report, in order to check leakage from the reservoir, first the south basin was taken out of use while the north basin remained in service, and then the north basin was taken out of use while the south basin remained in service. During the period the basins were out of service, or isolated, readings of the basin levels were made. The report states that during the period the south basin was isolated, a total of 8 days, the level within the basin dropped 2.79 feet which corresponds to an average loss rate of 0.526 MGD. Tests were also made during a period when the level of the in-service (north) basin was below the isolated (south) basin, and also when the level of the in-service (north) basin was above the level of the isolated (south) basin. After applying several factors to account for transmission of flow between the two basins, it was concluded that the calculated rate of leakage from the south basin was about 0.475 MGD. Water level readings taken during the test period illustrating the rise and fall of the north basin and the loss of water from the south basin are shown on Plate 13. A similar test was performed at the north basin. According to the report, the test period for the north basin was 16 days, the level dropped 1.18 feet, and the average rate of loss was 0.115 MGD. Adjusting this rate for transmission from the south basin, the calculated rate of leakage from the north basin was about 0.093 MGD. Water level readings taken during the test period illustrating the rise and fall of the south basin and the loss of water from the north basin are shown on Plate 14.

The report points out that the leakage from the south basin is about 4-to-5 times as great as the leakage from the north basin. Mr. Kammerer theorizes that an underground drainage system allows the leakage from the south basin to dissipate into the surrounding park soil (more likely one, or more, old filled-in sink holes) without causing damage to the embankment slopes, whereas the north basin has no such subsurface drainage system, and as a result, water from the basin is surfacing on the north slope and causing some erosion problems. The author concludes that construction of the new highway, I-44, is responsible for disturbing the natural underground drainage, which has resulted in some erosion damage to the north slope.

In the second section of the report, Mr. Kammerer presents the findings of inspections of the interiors of the north and south basins with respect to the structural condition of these basins. Due to time limitations, only a very cursory inspection of the south basin was made, and, except for indicating that no problems of major proportions were noted, no details of structural conditions are given. However, according to the report, the north basin was extensively inspected and surveyed.

The writer mentions that several floor slabs along the west side of the reservoir were found to be uplifted up to 4 inches, and cracked. These uplifted slabs were removed, repairs were made to the subpavement including the waterproofing membrane, and new slabs were installed. The locations of the new pavement slabs are shown on City Drawing 4532, reference Plate 15.

The report states that the division wall was found to be in sound structural condition, although water leaked through the expansion joints and at cracks in the wall. Leakage was also noticed at the sluice gates in the two gate chambers, and at the sewer gate. The stem of the sewer gate was found to be slightly bent.

The author states that the slopes of the basin were in good condition, although in some places the pavement joint material was in bad shape and repairs had to be made. The method of repair is described in the report. The interior side of the perimeter wall was found to be in good structural condition; however, the exterior side was found to be spalling badly. The

roof appeared to be structurally sound, with the exception of some scattered spalling where the reinforcing bars were exposed. Mr. Kammerer states that the roof beams in general looked good, although a few beams near the top of the slope where they could be more closely examined, were cracked.

The report indicates that the most obvious problem concerns the columns at the edge of the roof, which were constructed abutting the vertical wall about the basin. In order to allow for expansion and contraction of the roof system, brass plates were cast in the top of the columns and the underside of the roof beams at these columns. The author theorizes that the brass plates bind during the contraction cycle of movement, and points out that the columns furthest from the center of the structure have moved the most, with displacement at much as 7 inches observed. Since a similar inspection of the basin was made by Water Division personnel in 1970 at which time, according to individuals participating in the inspection, no significant movement of these columns was noticed, Mr. Kammerer is of the opinion that the condition has worsened considerably since 1970. (Having given this premise more thought since writing the report, Mr. Kammerer now believes that movement of these columns was probably overlooked during the 1970 inspection.) The relationship between the I-44 construction which took place about 1970, and the movement of the columns is considered; however, no conclusions are drawn to indicate the highway construction to be a contributing factor. A recommendation is made that correcting the problem with the columns, as endorsed by Sverdrup & Parcel in their report, be given the highest priority. The report also points out that most of the columns within the basin show signs of distress, some more so than others.

In conclusion, the author states that lining of the basin will halt the leakage, but that it would be very costly to do so, and that it is best to do the structural repair work at this time and the lining at some future time, although lining of the basin would be beneficial to the outside walls, since it would stop the flow of water through the wall which is contributing to spalling of the surface on the outside of the wall. It is also recommended that a more detailed inspection of the south basin be made at the earliest opportunity, and especially the columns and roof system in order to see if conditions similar to that found at the north basin exist.

Elevations of column footings, measurements of movement between columns and roof beams, measurements of gaps between end columns and walls, and between the slope pavement and wall, are shown on Water Division Drawings 4532 and 4533, reference Plates 15 and 16.

In the third section of the report, Mr. Kammerer presents the findings at a series of excavations made to check the condition of the footing, subgrade, etc., of the vertical wall at the north end of the reservoir. The locations of these excavations are shown on Drawings 4532 and 4533, Plates 15 and 16, and a schedule indicating the size of each is presented on Drawing 4533.

To summarize this section of the report, several of the excavations, there were 13 excavations in all, showed that the earth had settled up to 1.5 inches beneath the footing, a vertical crack was found in the footing of one of the excavations, a tension crack was found in one of the concrete piles supporting the footing, and two holes, one of which was about 2 inches in diameter, were found in the steel sheet piling. The writer states that the holes in the piling were sealed. During this time, the north basin was being refilled and it was necessary to pump water from the excavations. However, by directing the pump discharge down the stairs at the northeast corner of the reservoir, it was found that the north slope dried noticeably. Since it became apparent that sealing, or even finding all of the sources of leakage through the sheet piling would be nearly impossible, these exploratory excavations were halted. Mr. Kammerer reported that it was then decided to install the 4-inch diameter pipe subdrain system that runs parallel and adjacent to the north wall in order to intercept the leakage at this location. It is about this time that Water Division Drawings 4536 and 4537, reference Plates 17 and 18, were prepared showing the relationship of the north end of the reservoir, including the 4-inch diameter subdrain, and the highway.

According to Mr. Kammerer, during the period of January through April of 1975, certain repairs were made to the perimeter beams and columns of the north basin. These repairs consisted primarily of the installation of new expansion plates at the locations of the original brass plate type expansion plates at each of the columns adjacent to the outside wall of the reservoir structure. Mr. Kammerer reported that after relieving the beam load on the

column, the brass plate of the column was removed along with about 8 inches of concrete beneath the plate at the top of the column; a Teflon coated stainless steel plate with stainless steel anchors was grouted in place at the column; a similar Teflon coated brass plate was glued, using an epoxy type adhesive, to the old brass plate of the beam; and last, the temporary supports of the beam were removed and the beam was allowed to rest on the column. Mr. Kammerer also reported that no repairs were made to the expansion plates of the south basin since a recent inspection of the roof support system for this basin indicated the original expansion plates were functioning satisfactorily.

f. As a result of a slight slope failure of the north embankment slope that apparently occurred sometime in 1972, an investigation of the stability of the north slope was made during a two year period beginning in the spring of 1975. Subsequently, a report titled "Slope Stability Study of Compton Hill Reservoir", dated June 28, 1977, reference Charts 4-33 through 4-46, was prepared. A drawing showing the north slope of the reservoir embankment, piezometers installed within the north slope for the purpose of measuring ground water levels, the 4-inch subdrain pipe installed in 1973 by the Water Division, and the I-44 highway retaining wall, including the 18-inch diameter subdrain at the wall, is shown on Water Division Drawing 4545, reference Plate 19.

In the report, which was written by Stephen J. Runde, Civil Engineer II, Design and Construction Section, City of St. Louis Water Division, procedures for investigating the stability of the slope as well as the effectiveness of the 4-inch subdrain are described. The report states that seven piezometers were installed in the north slope in April of 1975 for the purpose of maintaining ground water levels. Based on readings of ground water levels, the author is of the opinion that water is flowing in a northeastwardly direction, that the 4-inch subdrain system is performing well, and that the presence of excessive ground water within the slope is eliminated. Mr. Runde states that since the soil is of high strength and the 4-inch drain is maintaining the slope dry, the slope is very stable provided that the filter material about the subdrain pipes is effective in preventing the transportation of soil solids. Samples of the flow in the 4-inch and 18-inch

pipes were obtained and tested. By inspection of the samples, it was determined that no suspended solids were present. Additional tests were made to check the amount of dissolved solids; however, these tests only indicated that the mineral content of the samples increased with the level of the reservoir, that the minerals found were not consistent with those present in the overlying soil, and even so, loss of these minerals did not affect the strength of the soil. Piping of the soil was therefore ruled out as a likely cause of slope failure.

In conclusion, Mr. Runde states that the slope is stable and that no further investigations are necessary at this time. It is recommended that the piezometers be monitored, flow measurements of the drains be made, samples of the flow be taken in order to check for solids, and that all of these items be done on a three to four month basis.

Records of flow and the results of total solid tests performed on samples obtained from the two subdrains for the period beginning June 30, 1975 and ending December 28, 1976 are shown on Charts 4-43 and 4-44. Records of piezometer ground water levels for the period beginning May 15, 1975 and April 29, 1977 are shown on Charts 4-45 and 4-46. Additional flow measurements for the period November 8, 1978 through June 29, 1979 and the period August 3, 1979 through February 6, 1981 are shown on Chart 4-47; additional piezometer readings for the period July 3, 1979 through February 6, 1981 are presented on Chart 4-48. R. E. Sauthoff, Civil Engineer, Horner & Shifrin, was present during the February 6th readings of piezometer ground water levels and subdrain flows. The flow periods shown on Chart 4-47 are the times in seconds required to fill a 2.5 gallon bucket, which for the February 6, 1981 readings for the 4-inch drain corresponds to about 8.1 gpm (11,660 gpd) and for the 18-inch drain, to approximately 26.5 gpm (38,160 gpd). Due to the fact that the flow readings for the 18-inch drain were taken at a manhole located downstream of the junction of the 4-inch drain, the values shown on Chart 4-47 and above for the 18-inch drain also include the flow in the 4-inch drain, and therefore, the actual flow in the 18-inch drain is the difference between the two readings, or in the above instance, 16.4 gpm (26,500 gpd).

According to the Stanley T. Fletcher, Division Engineer, Design and Construction Section, City of St. Louis Water Division, two sets of iron rods were set within the north embankment for the purpose of monitoring movement of the slope. The locations of these rods were checked periodically by the Water Division beginning on November 8, 1978 and ending on October 2, 1979, and records, reference Chart 4-49, were maintained. Some movement of the rods, up to 7/8-inch, is indicated. However, Mr. Kammerer is of the opinion that the rods may have been disturbed by equipment used to cut the grass as others obviously were and, therefore, the readings are somewhat questionable. The rod locations are no longer measured since all have either been disturbed or are missing.

4.3 MAINTENANCE OF OPERATING FACILITIES

According to Carl R. Schumacher, Division Engineer, Supply & Purifying Section, as reported by Joseph J. Kammerer, the gate operators for the manually operated sluice gates in the east and west gate chambers are serviced, but, because it is felt that other items have a higher maintenance priority, not on a regular basis. According to Thomas A. Rothermich, Division Engineer, Operating Section, the motorized sluice gate operators are remotely test operated once a month, and the sluice gates are fully closed once a year. Mr. Rothermich reported that the level transmitted by the telemetering equipment is checked daily by comparison with the level indicated by the staff gage located in the west gate house of the reservoir. Mr. Rothermich also indicated the high water alarm is checked once every 2 years, and that the full range of level sensing is checked every 5 years. Mr. Kammerer reported that the emergency gasoline operated generator located in the west gate house is test started once a month to insure its operation.

4.4 DESCRIPTION OF ANY WARNING SYSTEM IN EFFECT

According to Stanley T. Fletcher, Division Engineer, Design & Construction Section, the reservoir is routinely observed during the day by Water Division personnel as well as by employees of the Parks Department during the grass growing season, and by the City police patrolling the neighborhood area. Mr. Fletcher indicated that in case of an emergency, such as the imminent failure

of the reservoir embankment or wall, the remotely operated sluice gates controlling inflow to the reservoir would be closed and the sluice gates at the sewer chamber on the east side of the structure would be opened and the basins would be drained as quickly as possible. As previously indicated, the level of the reservoir is continuously monitored by a telemetering system, and a high level alarm device is provided.

4.5 EVALUATION

It is recommended that monitoring of the north slope of the reservoir embankment continue as planned. It is also recommended that, as soon as practical, repairs be made to the columns and beams supporting the north basin roof system, and that a detailed inspection of the interior of the south basin be made in the near future in order to determine its structural condition. In order to accomplish this inspection, it will be necessary to drain the basin, and, as stated in the Sverdrup & Parcel report, the basin should be lowered at a rate not to exceed 1-to-2 feet per day in order to avoid damage by hydrostatic pressure to the floor of the basin.

The slope stability study of the north embankment of the reservoir, performed in 1977 by Stephen J. Runde, is considered to be of limited value since the stability of the slope is evaluated in terms of past performance, i.e., Mr. Runde argues that since conditions relating to the slope, such as the level of the ground water table, have improved or in the case of the two subdrains, are as good as, or better than, that existing before the highway was constructed, the slope is stable. Although this reasoning has logic, it does not consider the fact that the stability of the slope could be marginal and that the factor of safety is borderline. As indicated in Section 2, paragraph 2.1, an extensive investigation of the north side embankment was made by the Missouri State Highway Department prior to the construction of Interstate Highway 44. However, this investigation was primarily for the purpose of determining the stability of the retaining wall to be constructed along the south side of the highway adjacent to the reservoir, and did not consider the possibility of failure of the reservoir embankment at levels higher than the wall, such as at the level of the bottom of the piling that supports the reservoir wall, or local failure of the slope itself, such as was experienced in 1972.

SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1 EVALUATION OF FEATURES

a. Design Data. Design data pertaining to the hydraulics of the reservoir were unavailable.

b. Experience Data. The level of water contained within the reservoir is unaffected by precipitation and/or storm water runoff. As previously indicated, the level of the reservoir is dependent upon user demand and leakage. The reservoir is supplied with potable water by pumping from the City's Chain of Rocks Treatment Plant. However, it can also be supplied, through a number of bleeder valves at cross connections, by the Stacy Park Reservoir. Reservoir inflow-outflow varies and no metering of flow at the reservoir is made.

c. Visual Observations.

(1) The reservoir is divided into two identical basins by a centrally located division wall. Each basin contains approximately 135.0 acre-feet (44.0 MG) of water at normal high water level, and 140.5 acre-feet (45.75 MG) of water at the overflow level. The basins are normally operated in parallel, but can be independently operated via a system of gates.

(2) Two openings, one for each basin, each approximately 1.4 feet high by 6.0 feet long, located in the sewer well of the east gate chamber near the top of the structure, is provided for basin overflow in the event of surcharge in excess of the maximum high water level. The level of the reservoir is continuously monitored and the surface elevation is transmitted by a telemetering system to both the Chain of Rocks Plant and the Kingshighway Avenue Office of the Water Division. Normal high water level is about 1.2 feet below the crest of the two overflow openings. An alarm is sounded at the Chain of Rocks Plant and at the Kingshighway Avenue Office in the event the reservoir level exceeds the normal level by 1.0 foot.

(3) A 20-inch diameter pipe sewer is provided to drain the sewer well at the east gate chamber. A gate valve, normally open, is located on the sewer at a point approximately 68.5 feet due east of the wall about the perimeter of the reservoir. The drain line connects to the City's sewer system at a manhole located at the intersection of Compton Hill Place and Geyer Avenue.

d. Hydraulic Analysis. An investigation of the 20-inch diameter pipe sewer that serves to drain the reservoir was made. The purpose of this investigation was to determine the capacity of this outlet in order to make some judgement of its adequacy with respect to the capacity of the two openings located within the sewer well of the east chamber.

Investigations of the hydraulics of the 20-inch sewer indicated the capacity of the pipe to be approximately 30 cfs, since for flows greater than about 30 cfs, it was found that the hydraulic gradient exceeded the level of the top of the manhole located just downstream of the sewer valve. A profile of the 20-inch diameter showing the relative locations and elevations of pertinent features is presented on page B-2 of Appendix B. An investigation of the hydraulics of the two 1.4-foot high by 6.0 foot wide openings provided for overflow of the reservoir basins indicated, using a weir approach, that the two openings could pass flow up to about 50 cfs before the top of the opening was exceeded by the level within the reservoir. It was therefore concluded that the governing feature for draining the facility is the 20-inch diameter sewer.

If it is assumed that inflow to the reservoir is not suspended at the normal high elevation level (elevation 185.4), or at the high water alarm level (elevation 186.4), and if it is also assumed that the inflow to the reservoir is equal to approximately 50 percent of the pumping capacity of one of the distribution pumps located at the Chain of Rocks Treatment Plant, (the remaining 50 percent being dispersed by the distribution system), the net flow entering the reservoir and overflowing the two openings at the sewer well would then amount to approximately 31 cfs. Reservoir outflow of 31 cfs is less than the capacity of the two overflow openings and approximately equal to the capacity of the 20-inch pipe sewer. It was, therefore, concluded that the

sewer provided to drain the reservoir as well as the basin overflow openings are of adequate proportions in the event of failure to terminate reservoir inflow, and no revisions to these features are considered necessary. In addition, no hazardous or damaging consequences relating to releases through the overflow openings at the sewer chamber were reported by the Owner's representative.

As previously indicated, for sewer flow in excess of 30 cfs, the hydraulic gradient will exceed the level of the top of the manhole located just downstream of the sewer valve and some local flooding of the area at this location may occur. Flooding of this area is not expected to be of serious consequence due to the fact that the excess flow will be retained between curb lines of the adjacent (Compton Hill Place, Geyer Avenue) downstream streets and collected by the various curb inlets located along these streets. However, it is recommended that drainage of the reservoir basins be controlled in order to prevent surcharging the sewer outlet to the point that overflow of the manhole occurs.

Calculations of sewer capacity, overflow capacity, and assumed reservoir inflow, are presented on pages B-3 through B-6 of Appendix B.

SECTION 6 - STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations. Visual observations of conditions which adversely affect the structural stability of the reservoir are discussed in Section 3, paragraph 3.1c and in Section 4, paragraph 4.2.

b. Design and Construction Data. Readily available design data relating to the structural stability of the reservoir is limited to the details shown on the construction plans, to stability analyses of the reservoir walls and to investigations of the north side of the reservoir by the Missouri State Highway Department prior to construction of Interstate Highway 44. Records of construction data relative to the structural stability of the reservoir, such as tensile tests of reinforcing steel, concrete compression tests, and soil compaction tests, were unavailable. Seepage and stability analyses of the reservoir embankment comparable to the requirements of the "Recommended Guidelines for Safety Inspection of Dams" were not available, which is considered a deficiency. These seepage and stability analyses should be performed for appropriate loading conditions (including earthquake loads) and made a matter of record.

c. Operating Records. Records beginning in 1960 of the reservoir level are on file at the Water Division's Chain of Rocks Treatment Plant. Records of the reservoir level prior to 1960 have been discarded. Records of the ground water level at the piezometers in the north slope of the reservoir embankment, the flows in the 4-inch and 18-inch pipe subdrains at the north slope, and measurements of iron rods set in the slope are available at the Kingshighway Avenue office of the Water Division. This data is also included herein, reference Charts 4-47 through 4-49.

d. Post Construction Changes. Two major post construction changes have taken place since the reservoir was originally constructed in 1868-1870 that are considered to have a bearing on the structural stability of the reservoir. The first such change occurred in 1915-1916 when the reservoir was raised approximately 10 feet by the addition of a wall about the perimeter of

the structure. As indicated in Section 2, paragraph 2.1.d., the stability of the reservoir wall was investigated. However, an investigation of the stability of the earthen embankment about the reservoir was unavailable. The second change occurred in about 1969 when Interstate Highway 44 was constructed adjacent to the north side of the reservoir, and as a part of this construction, which is in cut, a retaining wall was constructed near the north embankment. As indicated above, an extensive investigation of the north side of the reservoir was made by the Highway Department to insure the stability of the highway retaining wall adjacent to the reservoir. Also, in 1933, a roof was added to cover the north and south basins of the reservoir. Since the structural condition of some elements of the roof system is somewhat suspect, failure of the roof is a possibility. However, it does not appear likely that such a failure would affect the overall structural stability of the reservoir.

e. Seismic Stability. The reservoir is located within a Zone II seismic probability area. An earthquake of the magnitude that might occur in this area would not be expected to cause structural damage to this facility provided that static stability conditions are satisfactory and conventional safety margins exist. However, it is recommended that the prescribed seismic loading for this zone be applied in any stability analyses performed for the reservoir embankment and wall.

SECTION 7 - ASSESSMENT/REMEDIAL MEASURES

7.1 DAM ASSESSMENT

a. Safety. As noted during the inspection, the exterior surface of the concrete wall about the reservoir is badly spalled in many areas. In some locations, the steel reinforcing bars at the top of the wall (not the principal steel) are exposed, and in other places, lime residue indicating the seepage of water through the wall was evident at the locations of cracks in the concrete. According to inspection reports by the Water Division, these structural defects have been apparent for some time.

Although not actually observed during the visual inspection, records provided by the Water Division, of previous inspections of the north basin of the reservoir indicated that many of the columns supporting the roof system are severely overstressed and deflected, in some instances adjacent to the outside walls of the structure, to the extent that very little support is provided the roof beams. Although it has been reported that repairs have been made to the perimeter column and beam expansion plates and that the area of bearing of the beam on the column has been increased somewhat, no judgement of the condition or effectiveness of these repairs can be made since an inspection of the reservoir interior was not made. However, it is known, as indicated above, that there are columns and beams within the north basin that are in poor structural condition and in need of repair.

Seepage and stability analyses of the reservoir embankments were not available for review, and therefore, no judgment could be made with respect to the structural stability of the embankment of the reservoir.

b. Adequacy of Information. The engineering data available for assessing the design and construction of the reservoir is considered adequate for the purpose of this investigation. As indicated above, seepage and stability analyses of the reservoir embankments comparable to the requirements of "Recommended Guidelines for Safety Inspection of Dams" were not available, which is considered a deficiency.

c. Urgency. The remedial measures recommended in paragraph 7.2 for the items concerning the safety of the dam noted in paragraph 7.1a should be accomplished within the near future.

d. Necessity for Phase II. Based on the results of the Phase I inspection, a Phase II investigation is not recommended.

e. Seismic Stability. The reservoir is located within a Zone II seismic probability area. An earthquake of the magnitude that might occur in this area would not be expected to cause structural damage to this facility provided that static stability conditions are satisfactory and conventional safety margins exist. However, it is recommended that the prescribed seismic loading for this zone be applied in any stability analyses performed for the reservoir embankment.

7.2 REMEDIAL MEASURES

a. Recommendations. The following actions are recommended.

(1) Obtain the necessary soil data and perform seepage and stability analyses of the reservoir embankments in order to determine the structural stability of the facility for all operational conditions. These analyses should include the surcharge imposed by the reservoir wall and the maximum hydraulic level of the reservoir. Seepage and stability analyses should be performed by a qualified professional engineer experienced in soils and foundation engineering. It is also recommended that the City obtain for their files a copy of the April 7, 1967, Missouri State Highway Department report of their investigation of the stability of the north side of the reservoir.

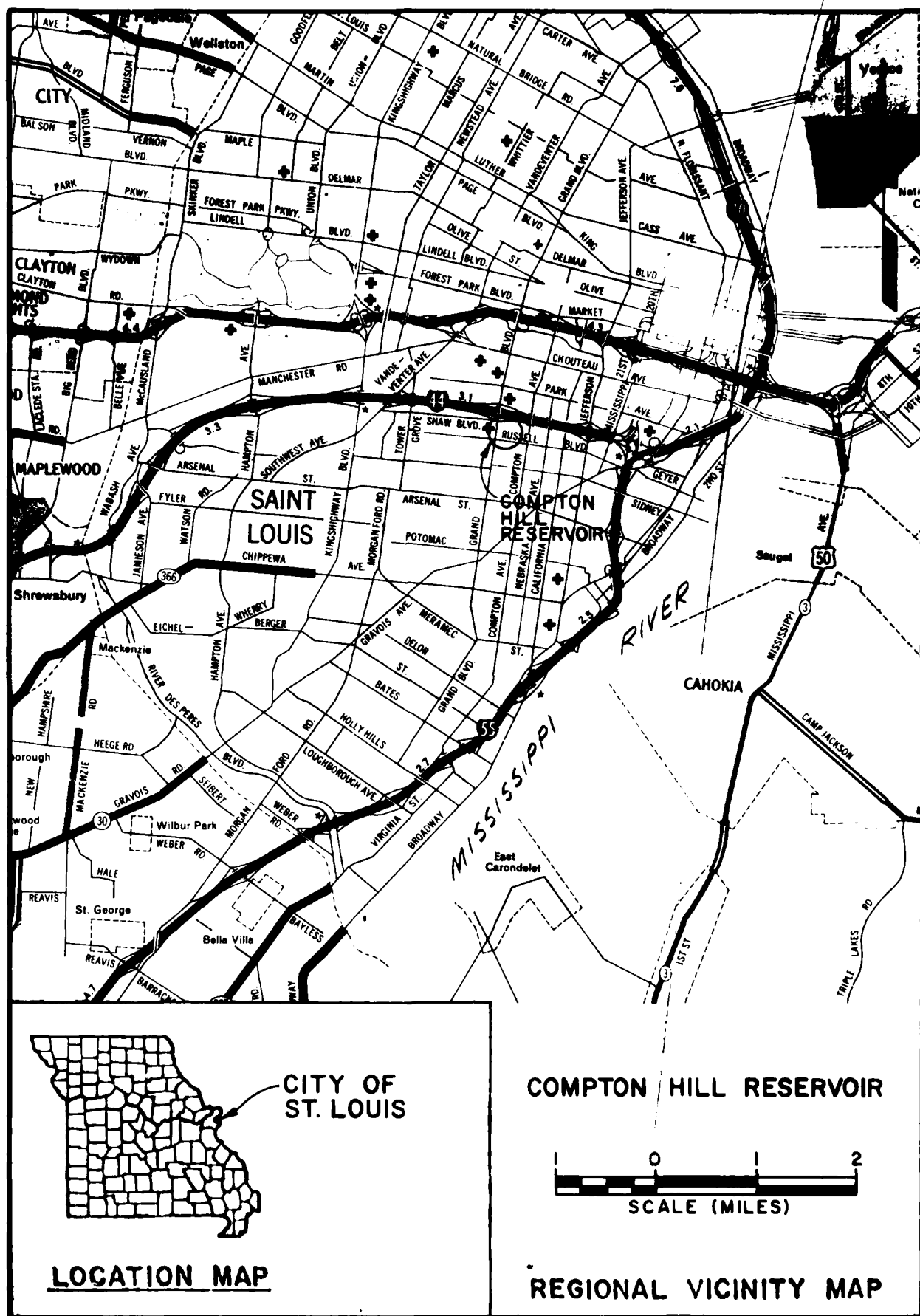
(2) The Owner should prepare contingency or emergency plans which provide for steps to be taken in the interests of public safety in the case of imminent structural failure of the reservoir. Due to the fact that the reservoir is located in a highly populated urban area, an emergency flood warning system is considered desirable to mitigate potential for loss of life.

b. Operations and Maintenance (O & M) Procedures. The following O & M procedures are recommended.

(1) Repair as necessary the deteriorated areas of the outside walls of the reservoir and provide some means, such as lining of the interior side, of preventing leakage through these walls. Continued deterioration of the concrete and/or reinforcing steel within the wall could ultimately result in structural failure of the wall. The wall should be waterproofed prior to restoration of the exterior surface since moisture seeping through the wall will only result in re-spalling of these repaired areas.

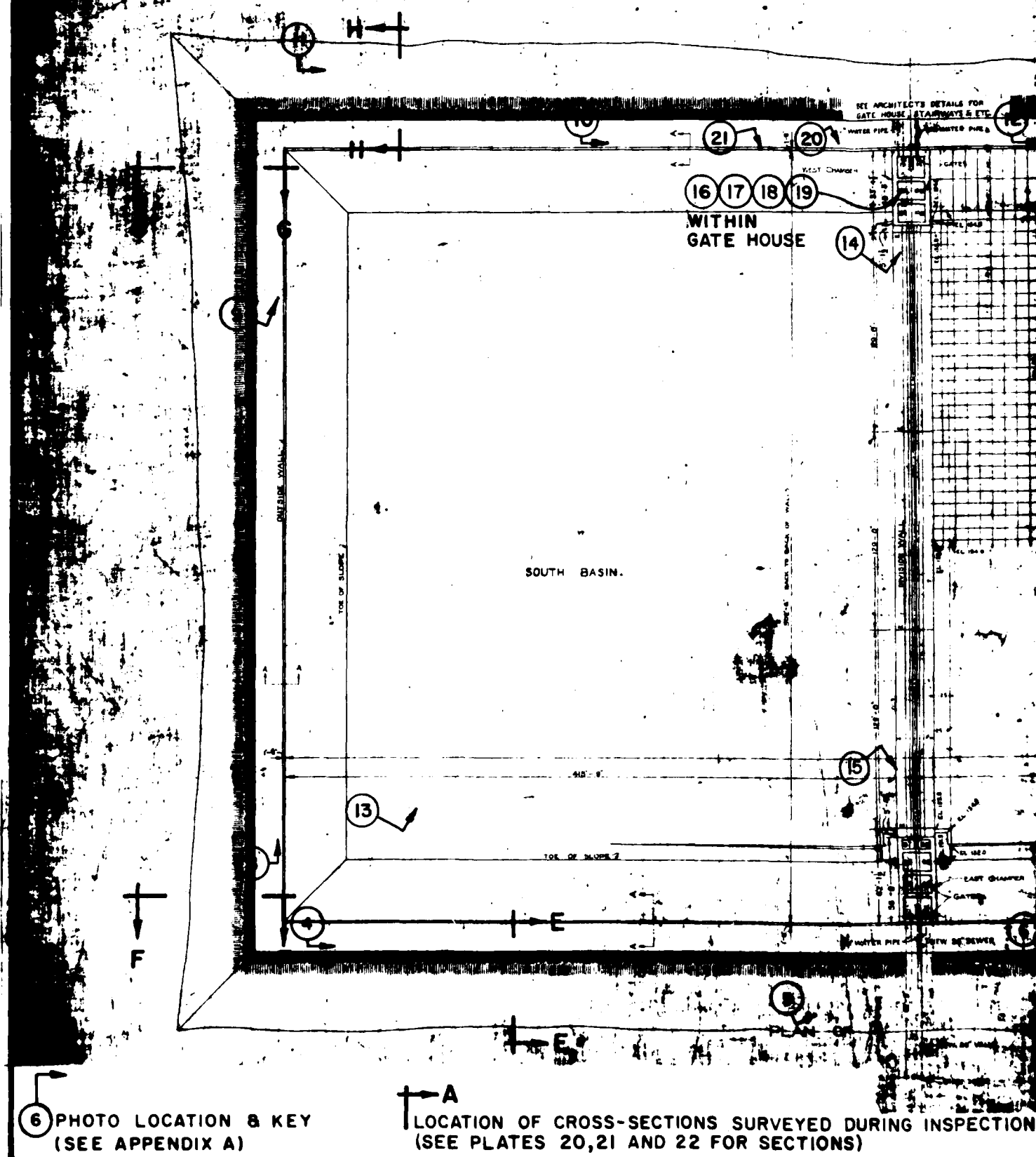
(2) Although not considered an item that might affect the safety of the reservoir, the Owner is advised to repair, or replace as necessary, the roof support beams and columns within the north basin of the reservoir. Loss of roof support due to beam or column failure could result in collapse of a portion, or all, of the basin roof.

(3) Since the inside of the north basin has not been inspected since 1975 when repairs to the column expansion plates were made, and since only a very cursory inspection of the inside of the south basin was made in 1973, it is recommended that a detailed inspection of the interiors of each basin be made sometime in the near future, and that similar such inspections be instituted on a periodic basis, such as every five years or so. It is also recommended as is the policy of the Water Division, that records be kept of all inspections made and remedial measures taken.

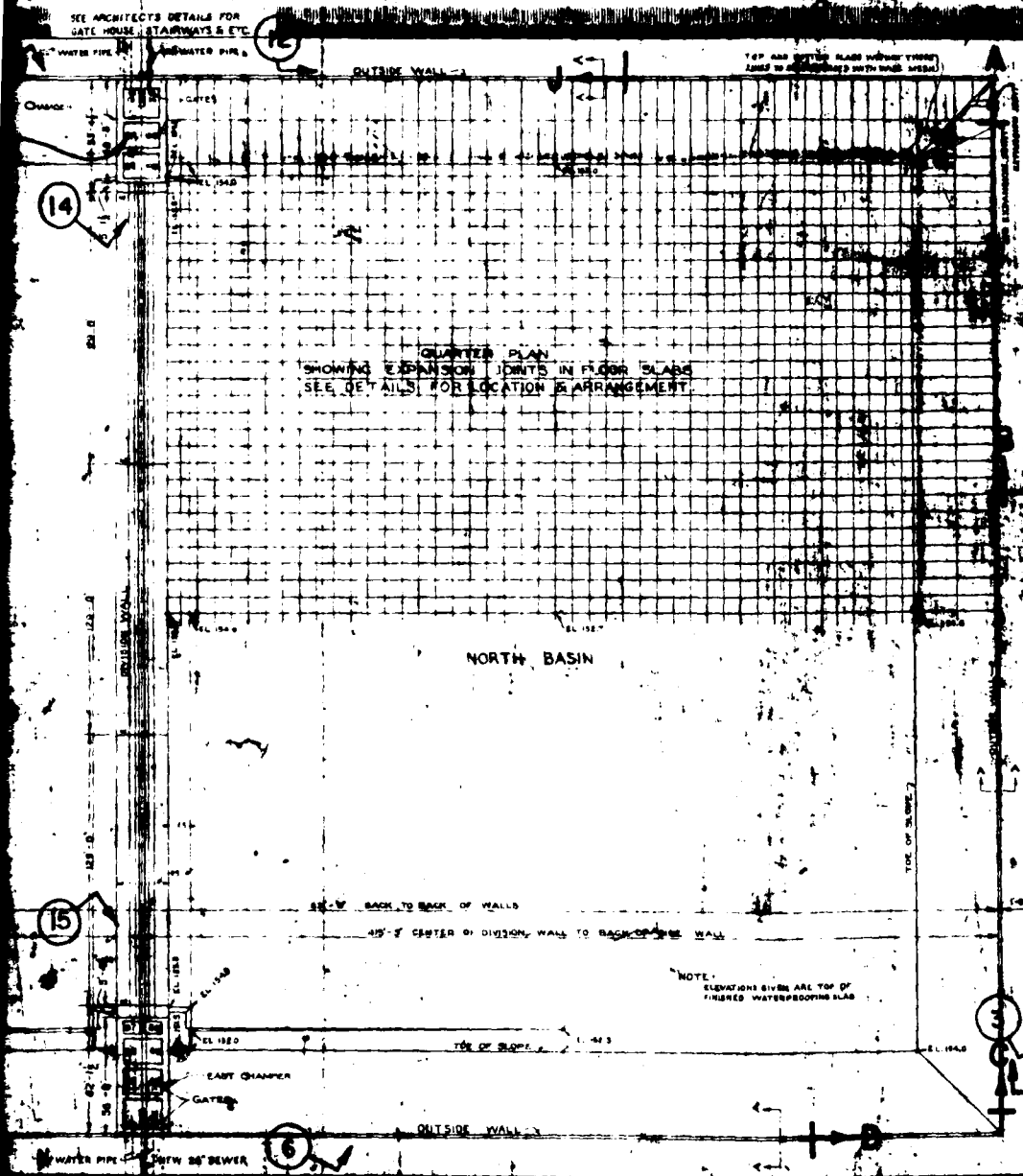




NOTE: THIS DRAWING SHOWS RESERVOIR POND
TO CONSTRUCTION OF ROOF.

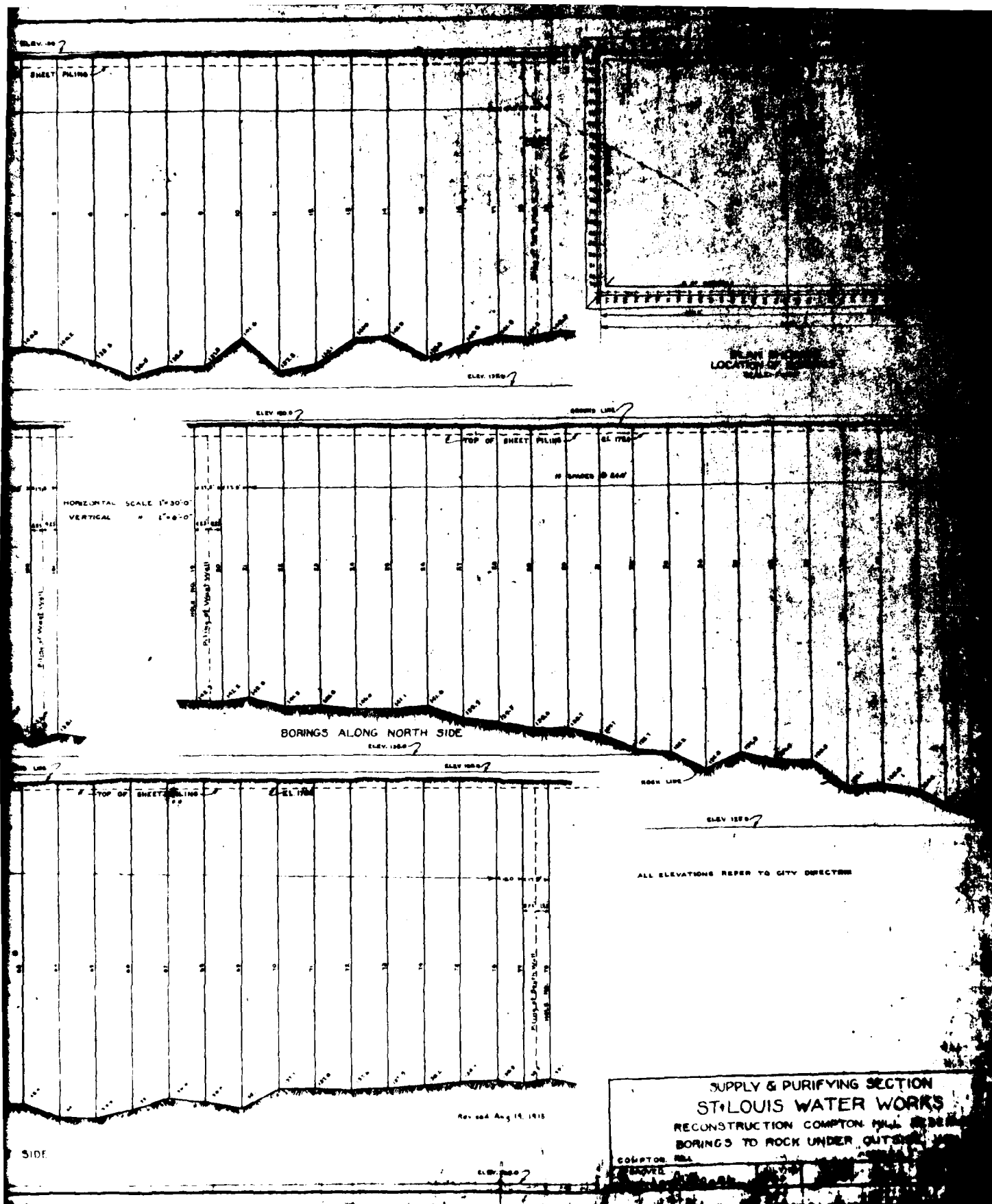


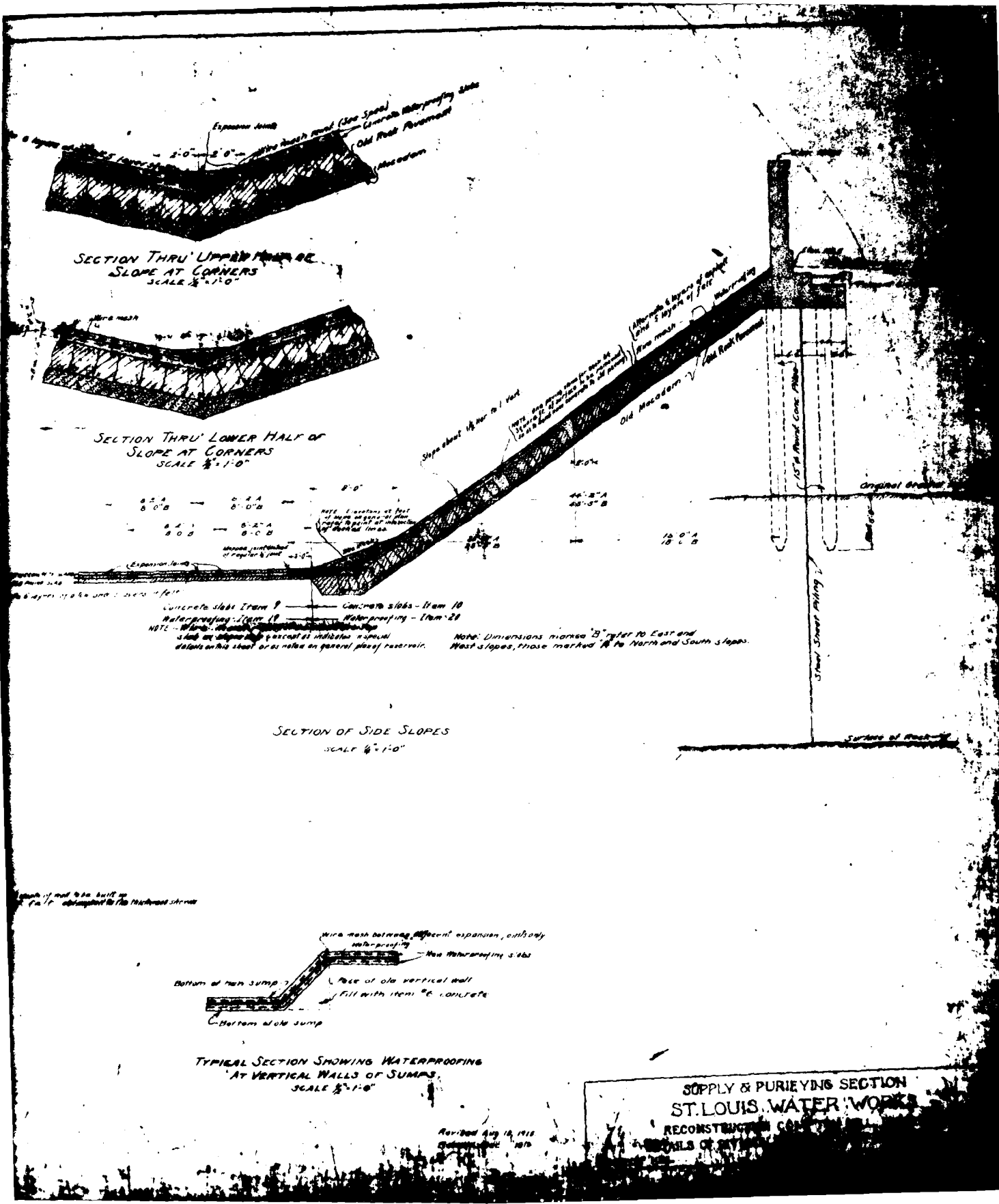
SHOWS RESERVOIR PRIOR
TO ROOF.



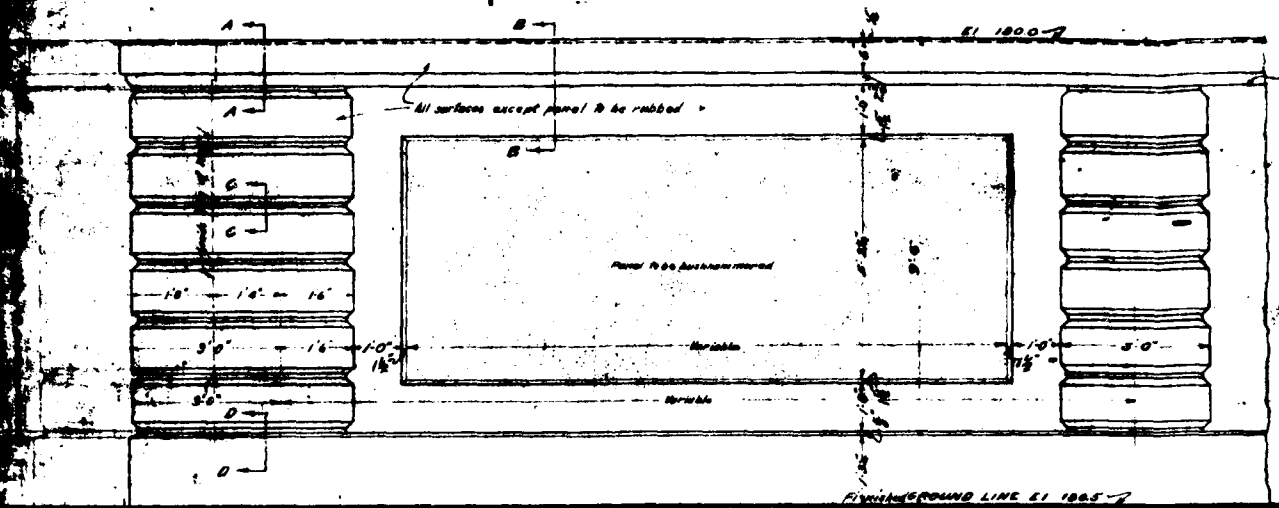
ED DURING INSPECTION OF 1-27-81.
(ONS)

12





SUPPLY & PURIFYING SECTION
 ST. LOUIS WATER WORKS
 RECONSTRUCTION CONTRACT NO. 1
 DETAILS OF REVISION



ELEVATION OF CORNER & SINGLE BAY OF WALL

DO NOT SCALE.



SEC. A-A



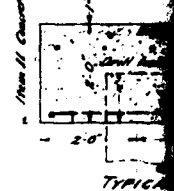
SEC. B-B



SEC. C-C



SEC. D-D



TYPICAL

$$24 \times 16.25 = 390.0$$

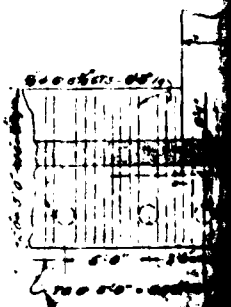
ONE HALF ELEVATION OF WEST WALL
SCALE 1/8" = 1'-0"

$$51 \times 16.3 \text{ (approx)} = 831.3$$

ONE HALF ELEVATION OF EAST WALL
SCALE 1/8" = 1'-0"

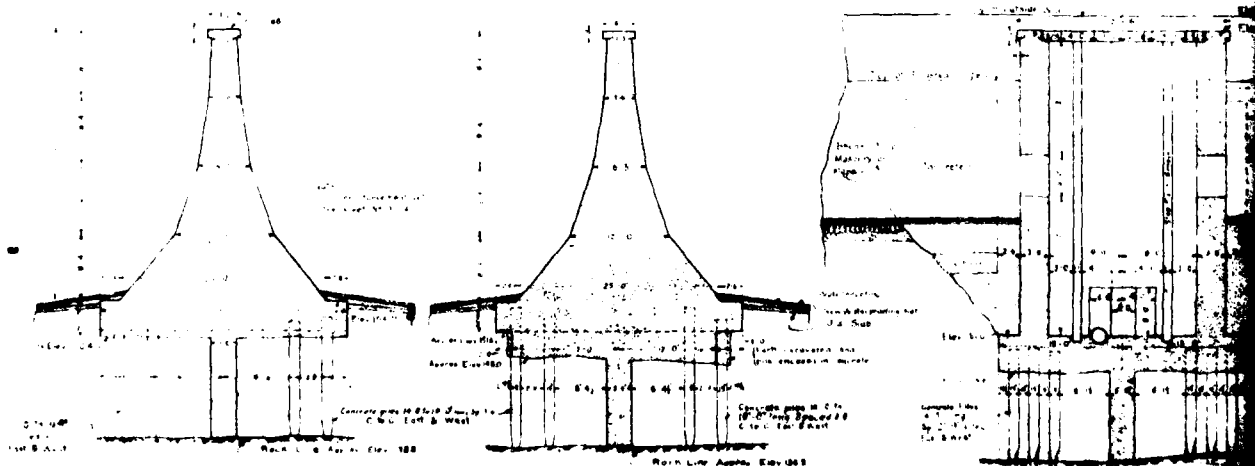
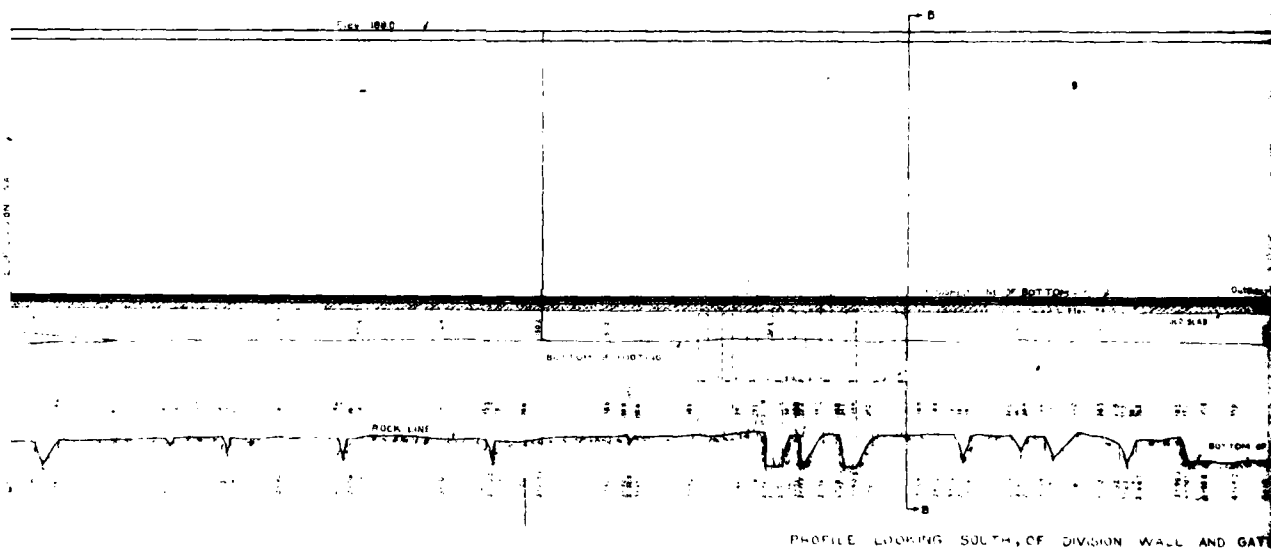
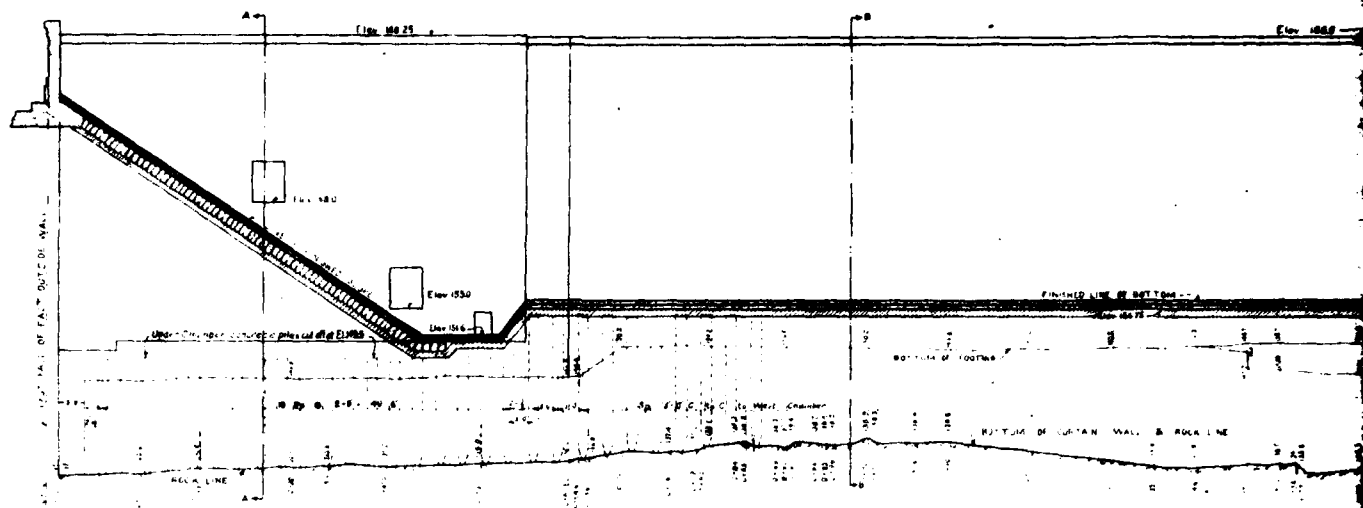
$$31 \times 16.1 \text{ (approx)} = 499.1$$

ONE HALF ELEVATION OF END WALLS
SCALE 1/8" = 1'-0"



PLAN OF WALL

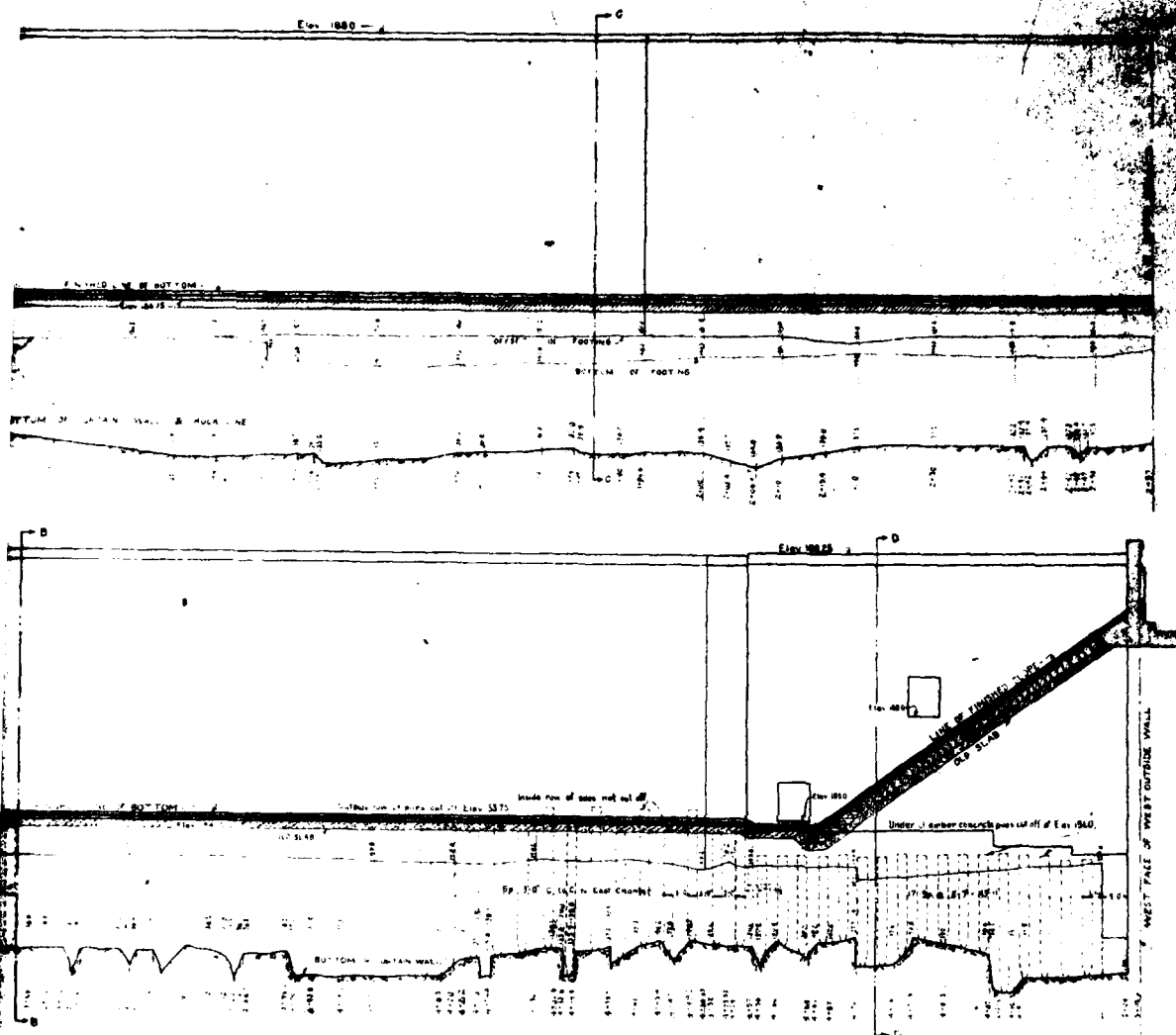
Note: The additional plan of West chamber are shown in plan.



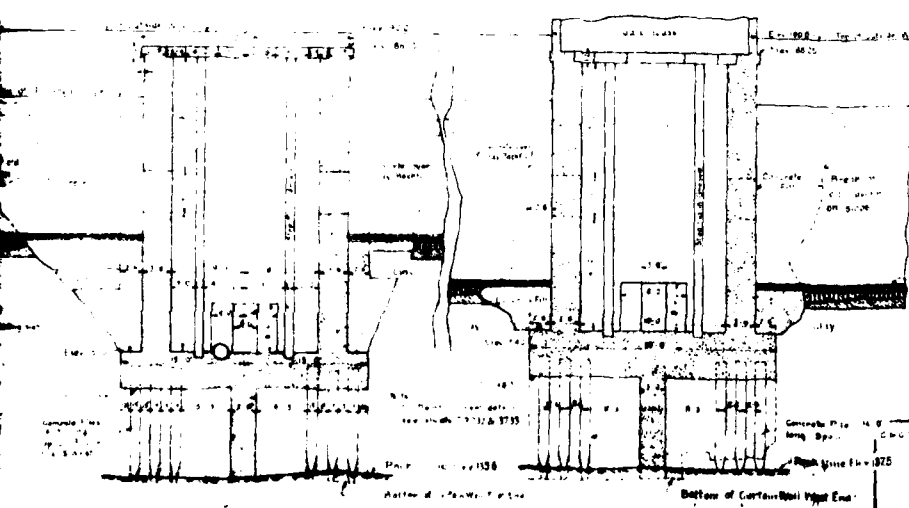
SEC-BR
Typical Cross Section From Sta 0+58.25
To Sta 2+70 Where Bottom Of Old Masonry
Of Old Masonry Wall Was Approximately Elev 1480

SEC-GG
Typical Cross Section From Sta 1+40
To 2+70 Where Bottom Of Old Masonry
Wall Was Approximately Elev 1480

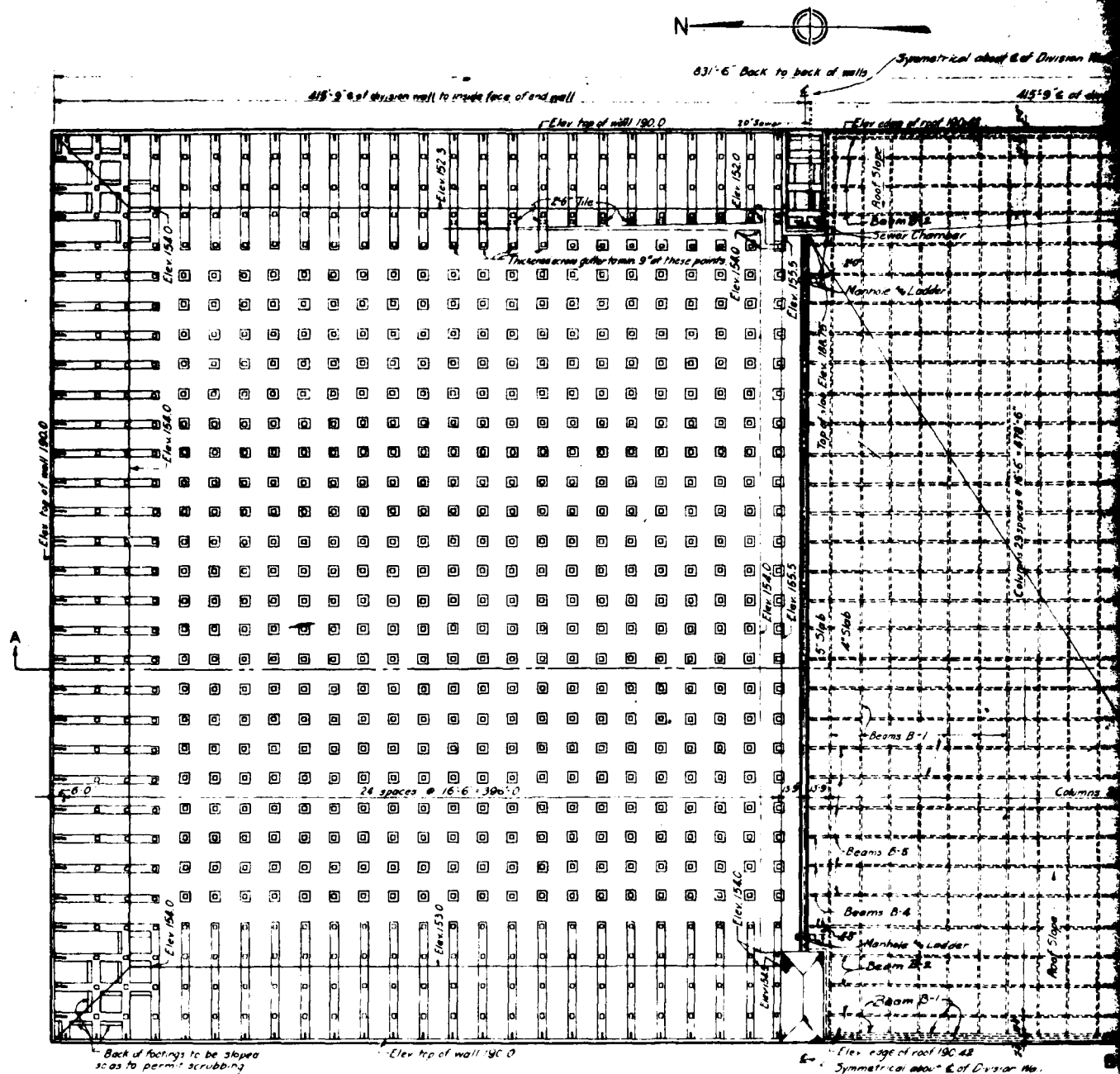
SEC-AA OF EAST CHAMBER



LOOKING SOUTH, OF DIVISION WALL AND GATE CHAMBERS AS BUILT



SUPPLY & PURIFYING SECTION
ST LOUIS WATER WORKS
RECONSTRUCTION COMPTON-PHELPS
DETAILS OF DIVISION WALL AND GATE CHAMBERS



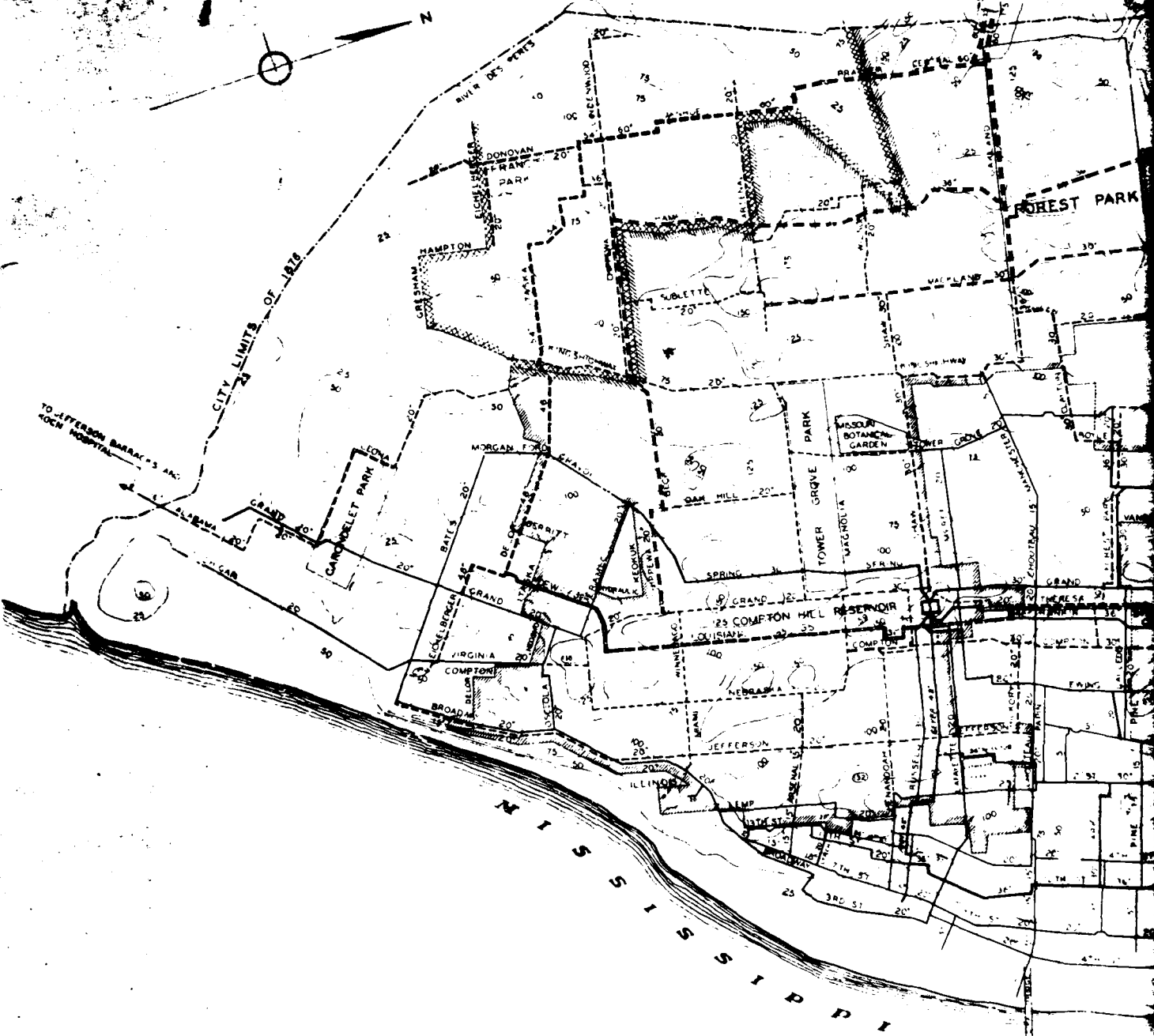
SECTION A-A

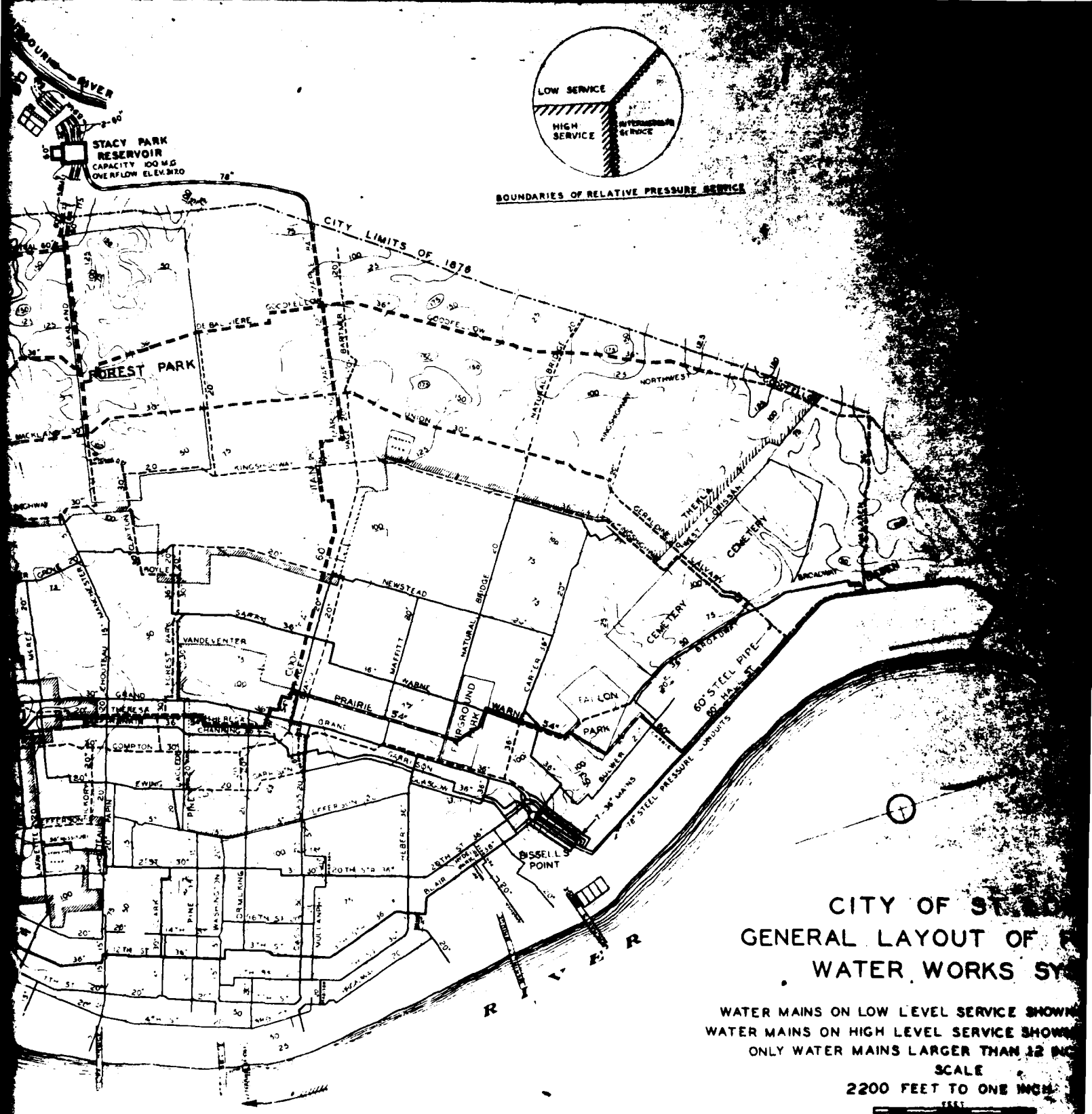
APPROVED BY BOARD OF SERVICE MEMBERS	
DESIGNED BY AIA	
DRAWN BY AIA	
CHECKED BY AIA	
DATE	
BY	
FOR	

HOWARD BEND
STATION



STACY PARK
RESERVOIR
CAPACITY 100 MG
OVERFLOW ELEV 170

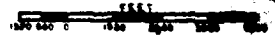




CITY OF ST. PAUL
GENERAL LAYOUT OF WATER MAINS AND SERVICE AREAS

WATER MAINS ON LOW LEVEL SERVICE SHOWN
WATER MAINS ON HIGH LEVEL SERVICE SHOWN
ONLY WATER MAINS LARGER THAN 12 IN.

SCALE
2200 FEET TO ONE INCH





CITY OF ST. LOUIS
LAYOUT OF PRESENT
WATER WORKS SYSTEM

LOW LEVEL SERVICE SHOWN AS SOLID LINES
HIGH LEVEL SERVICE SHOWN AS BROKEN LINES
MAINS LARGER THAN 12 INCHES ARE SHOWN

SCALE
2200 FEET TO ONE INCH

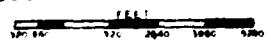


EXHIBIT A

MAY 1913
REVISED APRIL 1915
JUNE 1918

Howard Bond File Cabinet
T. H. Shaw, Engineer

PIPE SIZE

3"

6"

8"

10"

12"

15"

16"

20"

30"

36"

48"

STEEL

SADDLE

BLOW OFF

CAP - PLUG - OR
GOOSENECK PLATE

AIR TAP-PITO TAP

LINE VALVE

3 WAY

4 WAY

PATENT SADDLE & VALVE

PRESSURE REGULATOR VALVE

ANODE

SPRINKLING HYDRANT

CHECK VALVE

MANHOLE

FIRE HYDRANT

SLEEVE

VERTICAL CURVES

BY-PASS ON VALVE CASTING

BY-PASS AROUND VALVE

ELECTROLYSIS CALIBRATION
BOX

DISTRIBUTION

SYMBOLS

ST LOUIS WATER DIVISION

REVISED - 1938

1-11-65 J.P.

9-20
A-197

36

34

173

36

GRAND

AVE

CASTLEMAN

SHAW

DE TONTY

SEE 36A FOR ENLARGED SCALE

CITY OF ST. LOUIS WATER DIVISION
NOTE:
UNDERGROUND FACILITIES, SYSTEMS AND UTILITIES HAVE BEEN PLOTTED FROM AVAILABLE SURVEYS AND RECORDS AND, THEREFORE, THEIR LOCATIONS MUST BE CONSIDERED APPROXIMATE ONLY. THERE MAY BE OTHERS. FOR RELIANCE OF THESE IS POSSIBLE AT OWNERS RISK.

JAN 27 1981

SEE 36A FOR BLOW-UP

COMPTON

HILL

RESERVOIR

36

34

36

33

LOUISIANA

AVE

COMPTON

HILL

PLACE

RUSSELL

AVE

VIRGINIA

AVE

COPELIN

LAFAYETTE

COMPTON

AV

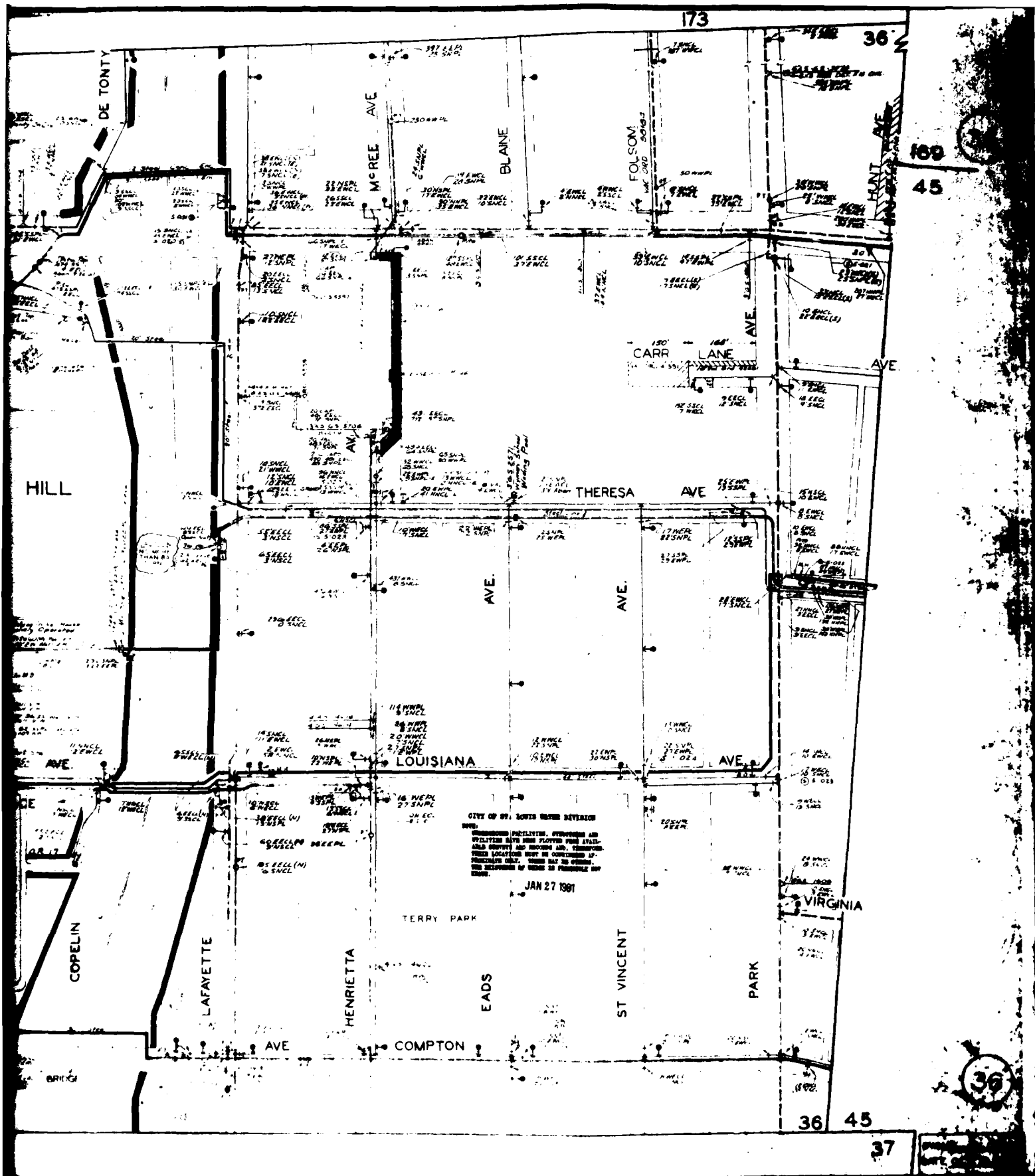
GEYER

33

36

37

ALLEN



34 36A

36

CASTLEMAN AVE

SHAW AVE

GRAND

POND

AVE

CITY OF ST. LOUIS WATER DIVISION
NOTE:
UNDERGROUND FACILITIES, STRUCTURES AND
UTILITIES HAVE BEEN PRINTED FROM AVAILS.
DATE SURVEYED AND LOCATED ANY TYPED
THEIR LOCATIONS MUST BE CONFIRMED AT
PROXIMATE ONLY. THERE MAY BE OTHERS.
THE DEPT. OF PUBLIC WORKS IS RESPONSIBLE FOR
THEM.

JAN 27 1981

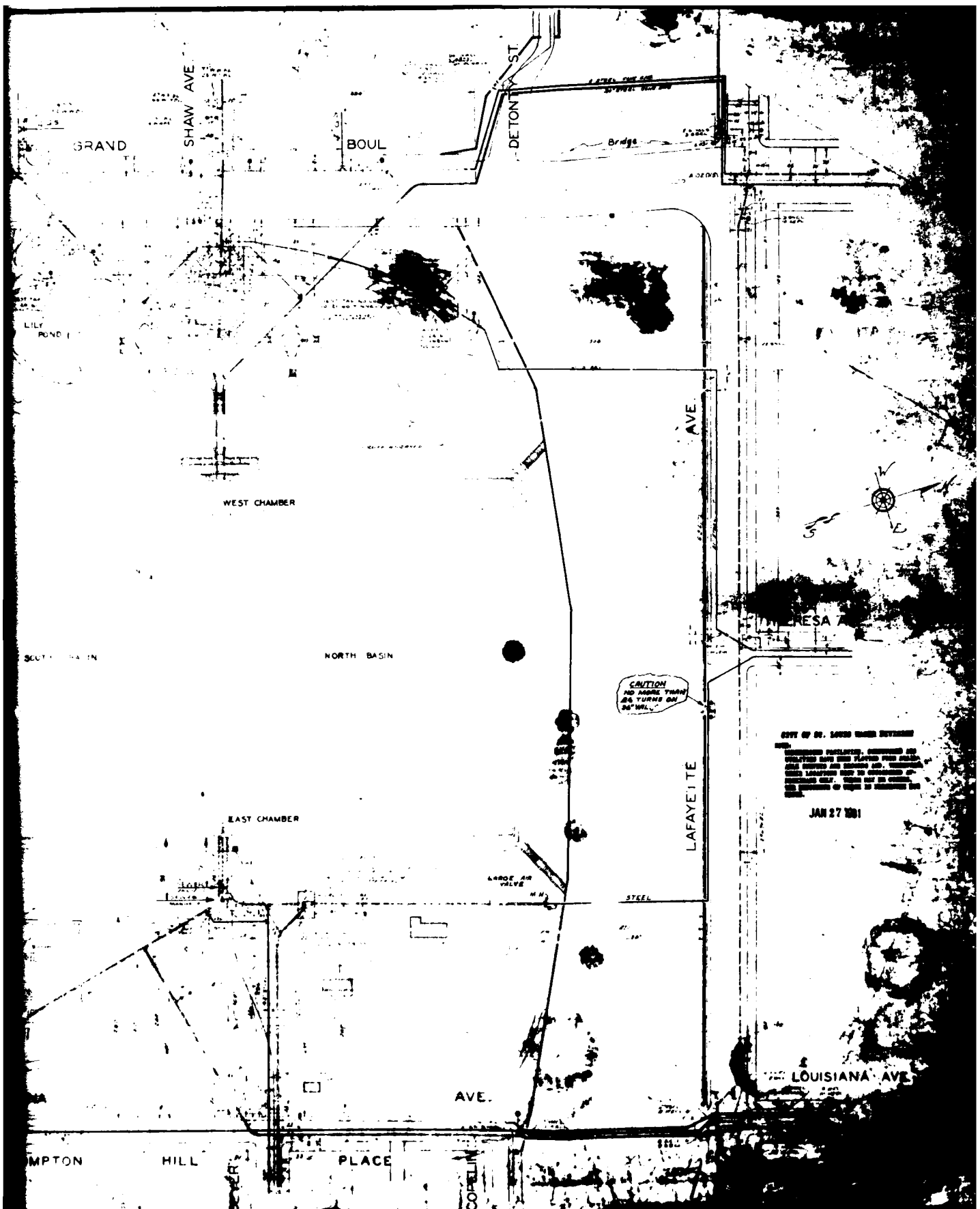
SOUTH SALIN

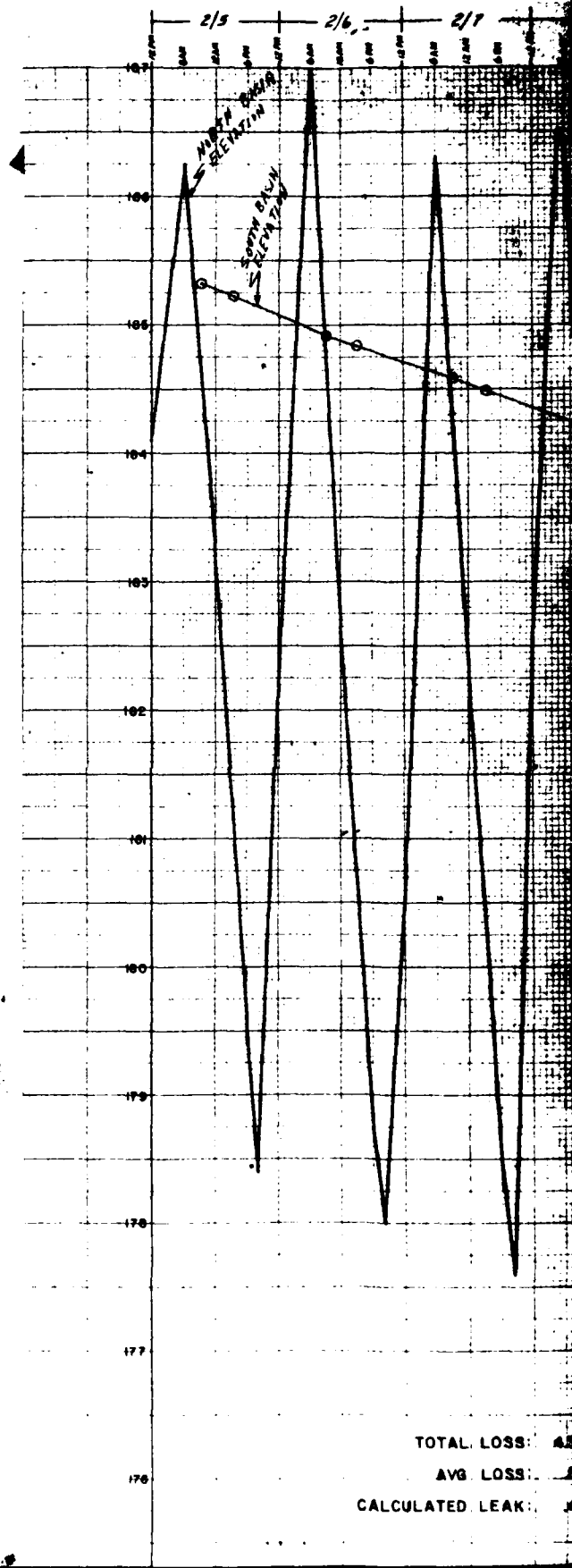
U.S.S.F.

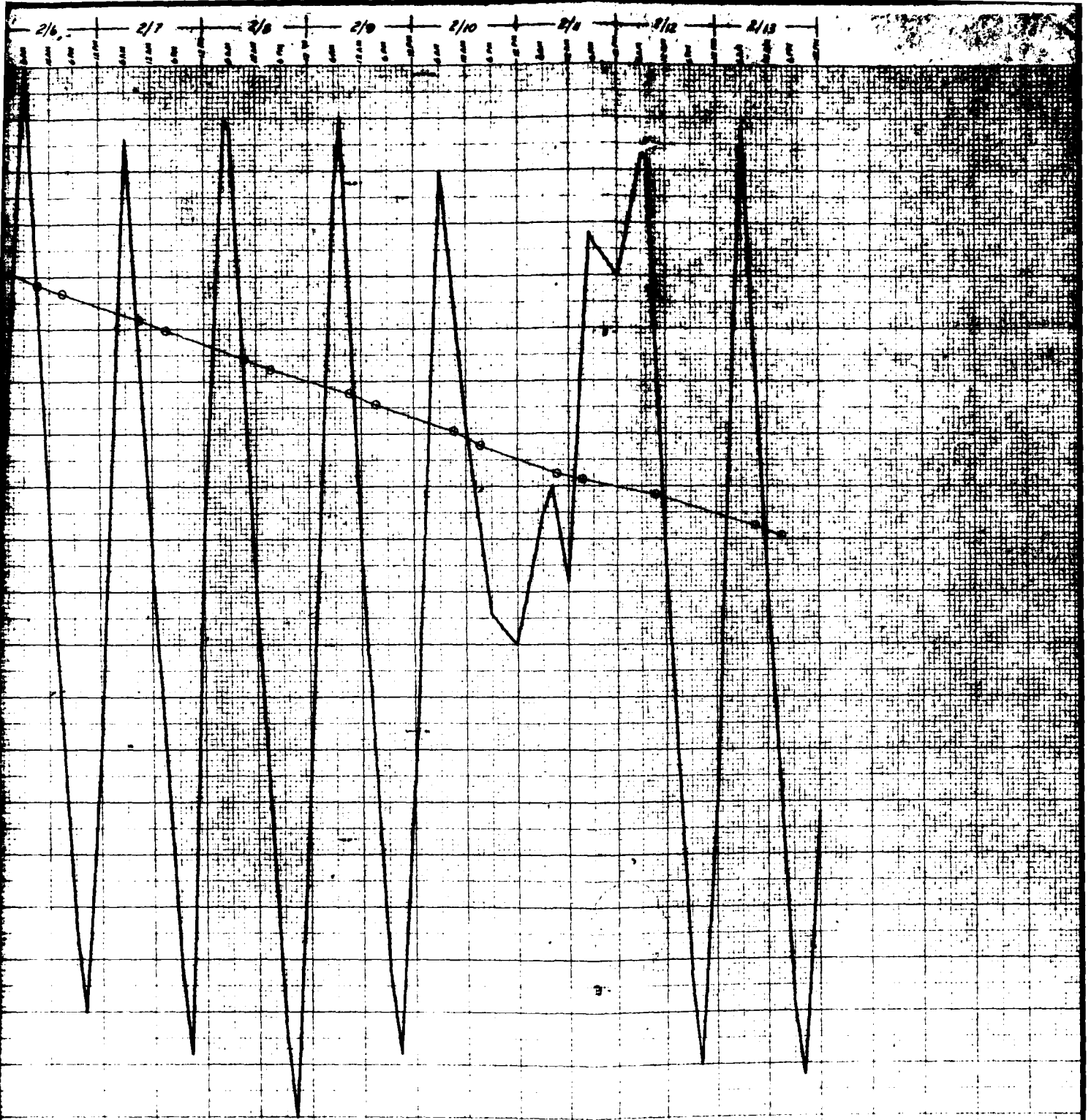
LOUISIANA

COMPTON

HILL







TOTAL LOSS: 4.34 MILLION GALLONS

AVG. LOSS: .526 MGD

CALCULATED LEAK: .475 MGD

LEAKAGE RESULTS
COMPTON HILL
SOUTH BASIN



TOTAL L
AVG. J
CALCULATED L



TOTAL LOSS: 1834 MILLION GALLONS

AVG. LOSS: .115 MGD

CALCULATED LEAK: .093 MGD

LEAKAGE RESULTS
COMPTON HILL
NORTH BASIN

26. 27. 28.

• • • • •

[illegible][illegible]

2 - 0 - N

I-44

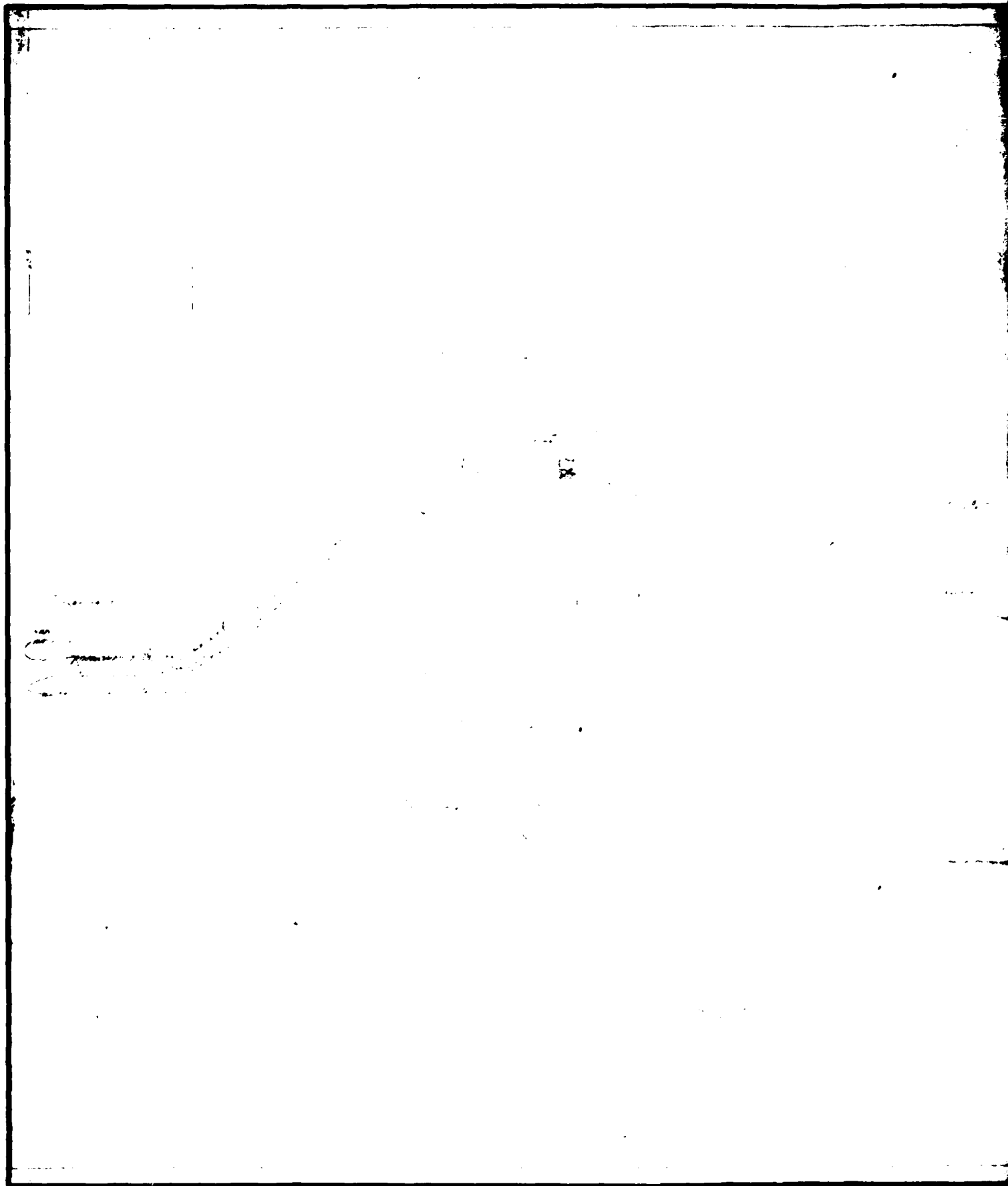
DESIGNED BY: J. W. ALLEN
 DRAWN BY: J. W. ALLEN
 CHECKED BY: J. W. ALLEN
 REVISIONS: 1. 10/1/44
 2. 10/1/44
 3. 10/1/44
 4. 10/1/44
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 100. 10/1/44

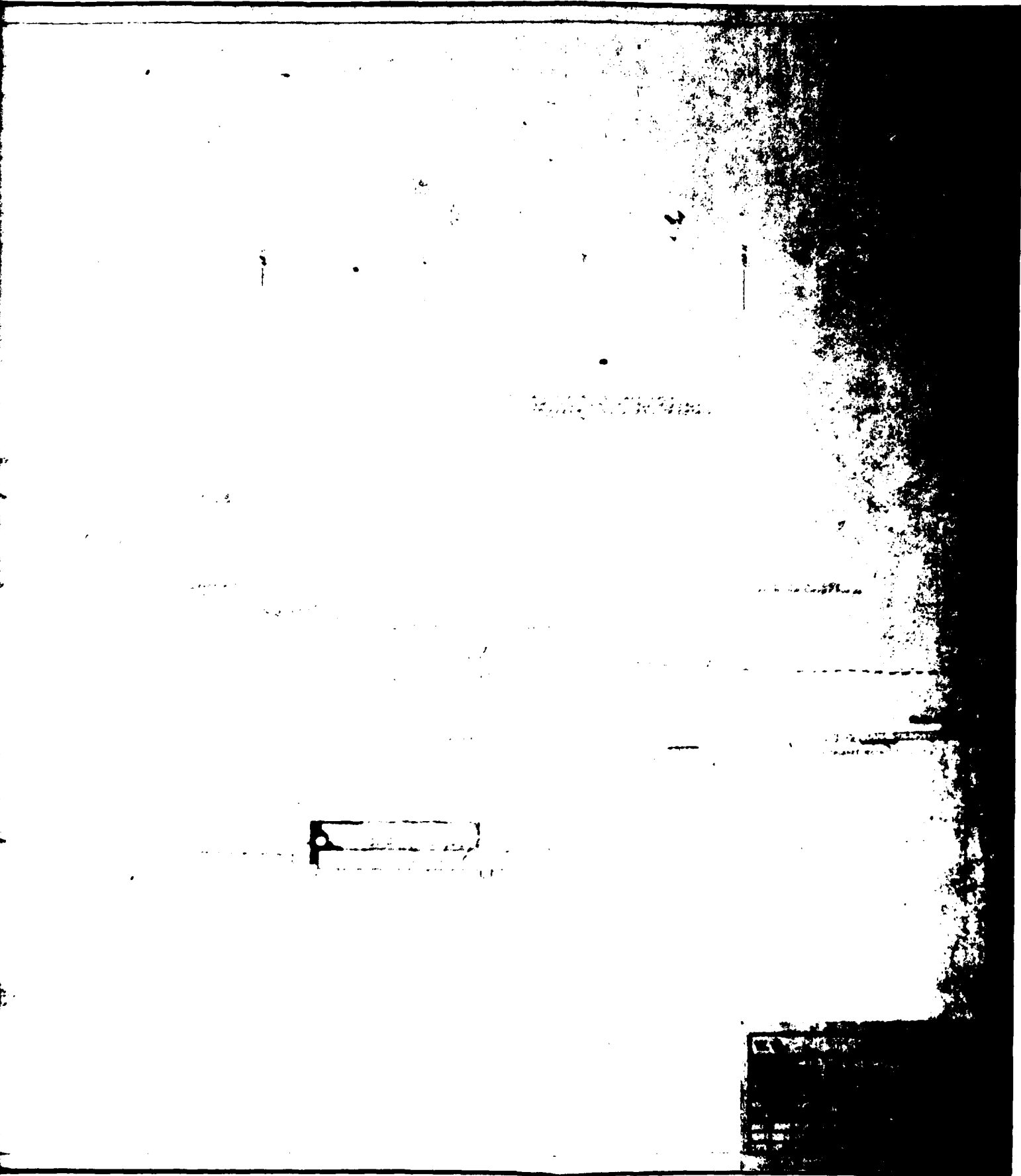
W.O. ALLEN		SHEET 1 OF 1	
DESIGNED BY	J. W. ALLEN	CHECKED BY	J. W. ALLEN
DRAWN BY	J. W. ALLEN	TRACED BY	J. W. ALLEN
CHECKED BY	J. W. ALLEN	REVISIONS	1. 10/1/44 2. 10/1/44 3. 10/1/44 4. 10/1/44 5. 10/1/44 6. 10/1/44 7. 10/1/44 8. 10/1/44 9. 10/1/44 10. 10/1/44 11. 10/1/44 12. 10/1/44 13. 10/1/44 14. 10/1/44 15. 10/1/44 16. 10/1/44 17. 10/1/44 18. 10/1/44 19. 10/1/44 20. 10/1/44 21. 10/1/44 22. 10/1/44 23. 10/1/44 24. 10/1/44 25. 10/1/44 26. 10/1/44 27. 10/1/44 28. 10/1/44 29. 10/1/44 30. 10/1/44 31. 10/1/44 32. 10/1/44 33. 10/1/44 34. 10/1/44 35. 10/1/44 36. 10/1/44 37. 10/1/44 38. 10/1/44 39. 10/1/44 40. 10/1/44 41. 10/1/44 42. 10/1/44 43. 10/1/44 44. 10/1/44 45. 10/1/44 46. 10/1/44 47. 10/1/44 48. 10/1/44 49. 10/1/44 50. 10/1/44 51. 10/1/44 52. 10/1/44 53. 10/1/44 54. 10/1/44 55. 10/1/44 56. 10/1/44 57. 10/1/44 58. 10/1/44 59. 10/1/44 60. 10/1/44 61. 10/1/44 62. 10/1/44 63. 10/1/44 64. 10/1/44 65. 10/1/44 66. 10/1/44 67. 10/1/44 68. 10/1/44 69. 10/1/44 70. 10/1/44 71. 10/1/44 72. 10/1/44 73. 10/1/44 74. 10/1/44 75. 10/1/44 76. 10/1/44 77. 10/1/44 78. 10/1/44 79. 10/1/44 80. 10/1/44 81. 10/1/44 82. 10/1/44 83. 10/1/44 84. 10/1/44 85. 10/1/44 86. 10/1/44 87. 10/1/44 88. 10/1/44 89. 10/1/44 90. 10/1/44 91. 10/1/44 92. 10/1/44 93. 10/1/44 94. 10/1/44 95. 10/1/44 96. 10/1/44 97. 10/1/44 98. 10/1/44 99. 10/1/44 100. 10/1/44
NO DATE BY		APPROVED	J. W. ALLEN
OFFICE OF DESIGN AND CONSTRUCTION WATER DIVISION DEPARTMENT OF PUBLIC UTILITIES CITY OF ST. LOUIS		SCALE 1" = 10'	

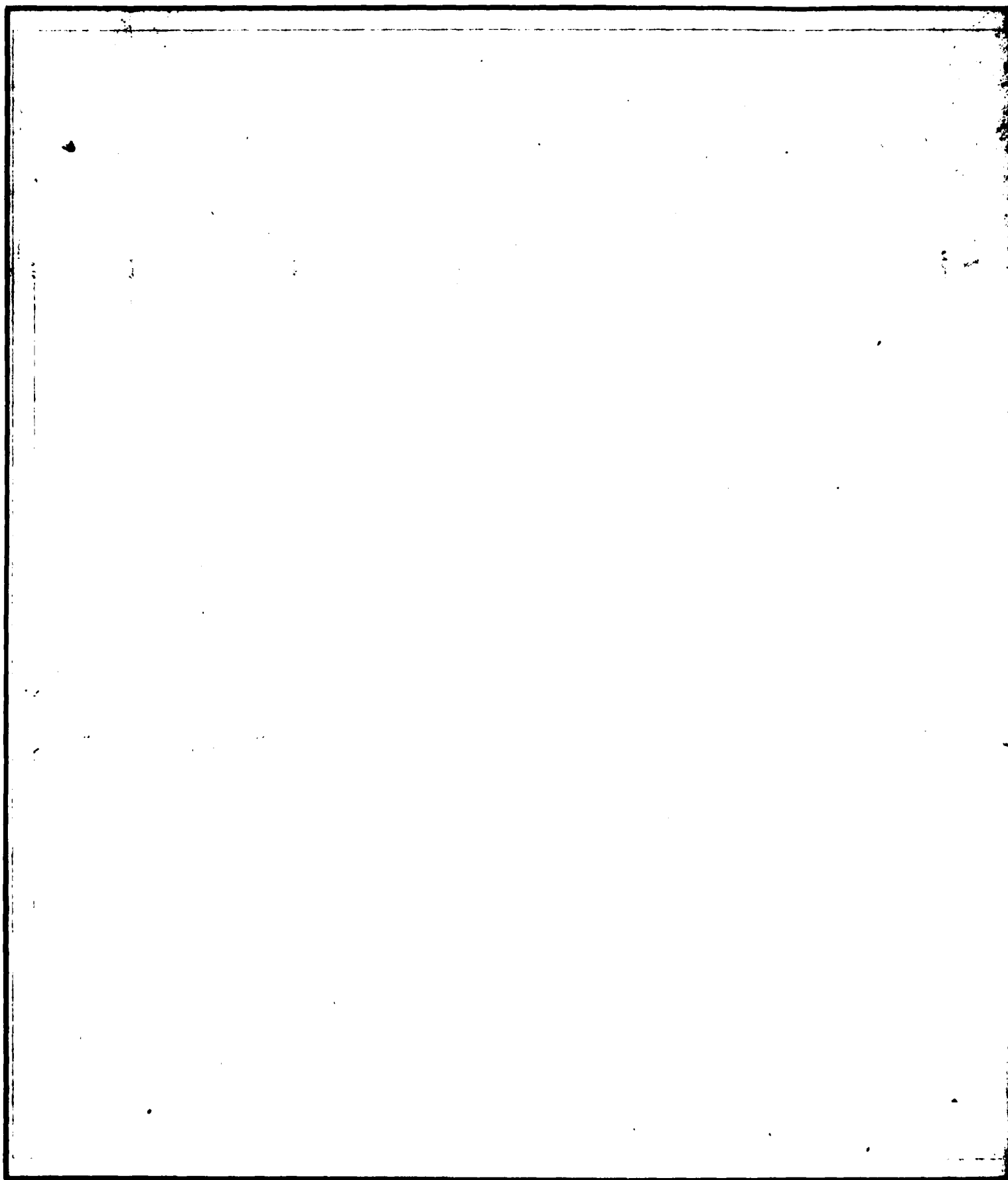
PLATE 15

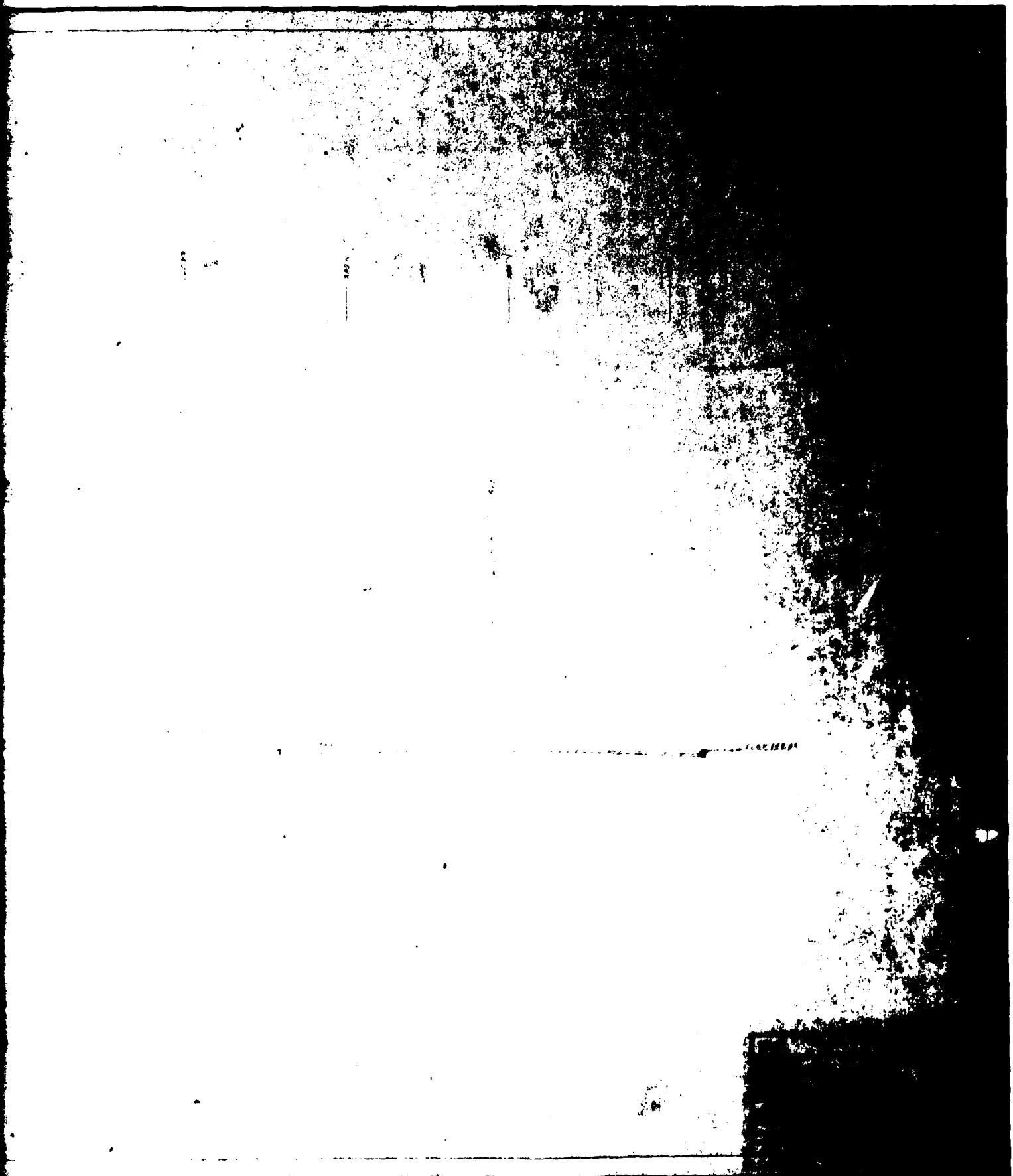
品名	單位	數量	價格	金額	備註
豬肉	斤	100	1.50	150.00	
牛肉	斤	50	2.00	100.00	
羊肉	斤	30	2.50	75.00	
雞肉	斤	80	1.20	96.00	
鴨肉	斤	40	1.80	72.00	
魚肉	斤	60	1.00	60.00	
蝦肉	斤	20	3.00	60.00	
蟹肉	斤	10	4.00	40.00	
蔬菜	斤	200	0.50	100.00	
水果	斤	100	0.80	80.00	
糧食	斤	500	0.20	100.00	
其他	斤	100	0.50	50.00	
合計				1000.00	

W.O. NO.		SHEET OF	
DESIGNED BY	DATE		
DRAWN BY	DATE		
TRACED BY	DATE		
CHECKED BY	OFFICE OF DESIGN AND CONSTRUCTION		
REVISED	WATER DIVISION		
NO DATE BY	DEPARTMENT OF PUBLIC UTILITIES		
	CITY OF ST. LOUIS		
APPROVED	SCALE	FILE NO.	
		DRAWING NO. 233	



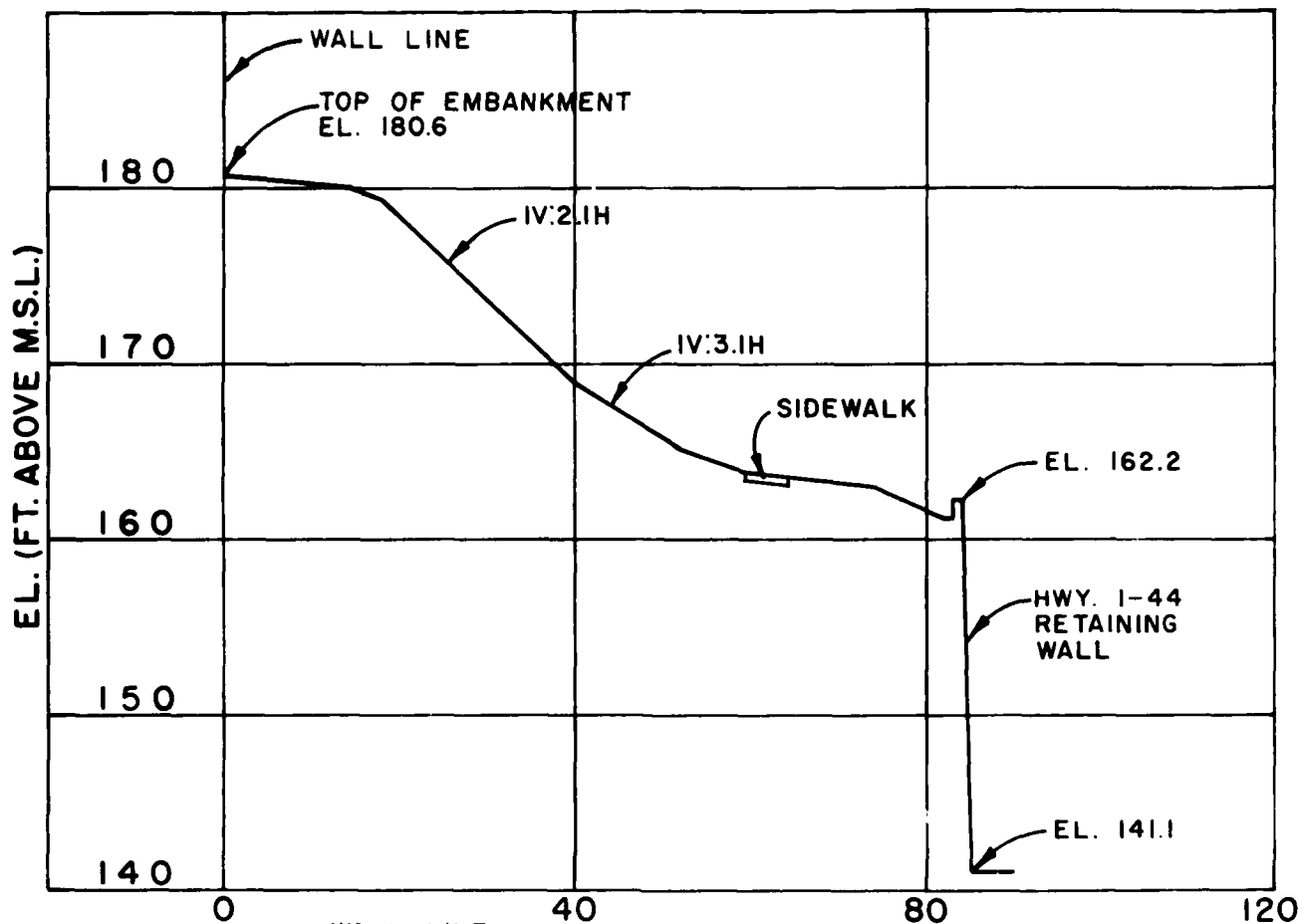






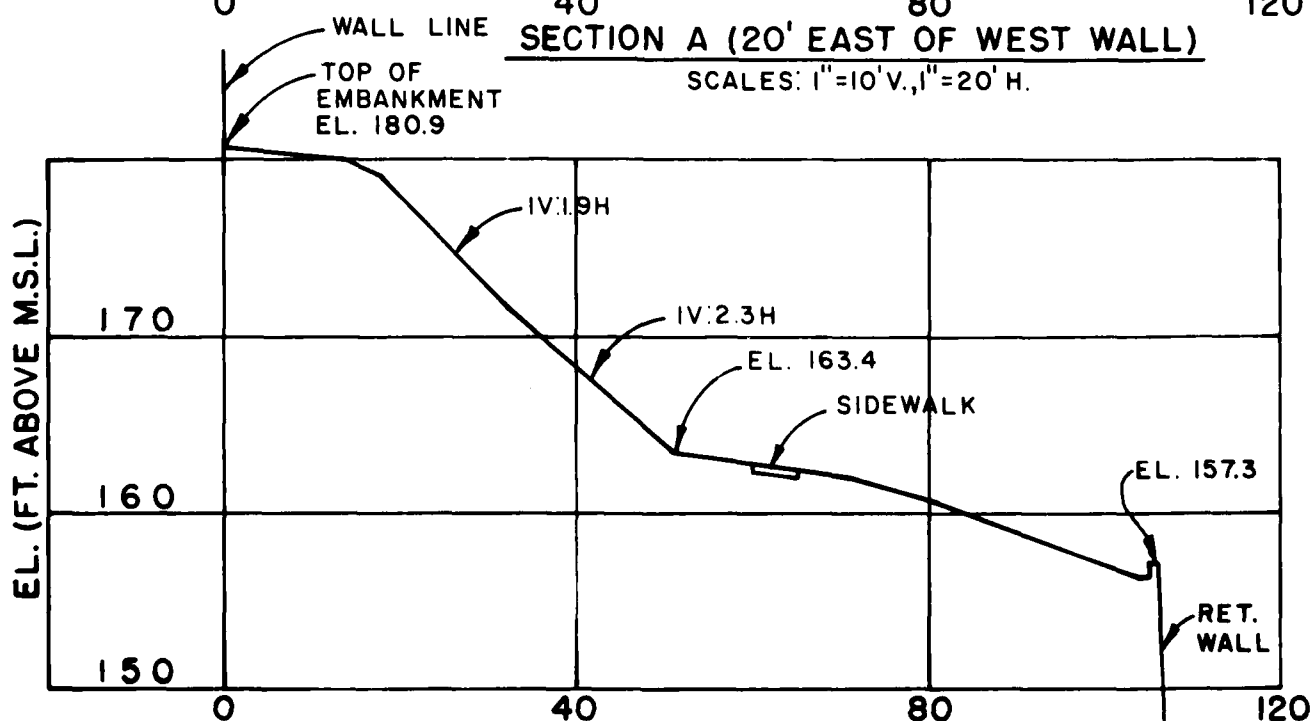
12

PLATE 18



SECTION A (20' EAST OF WEST WALL)

SCALES: 1"=10' V., 1"=20' H.



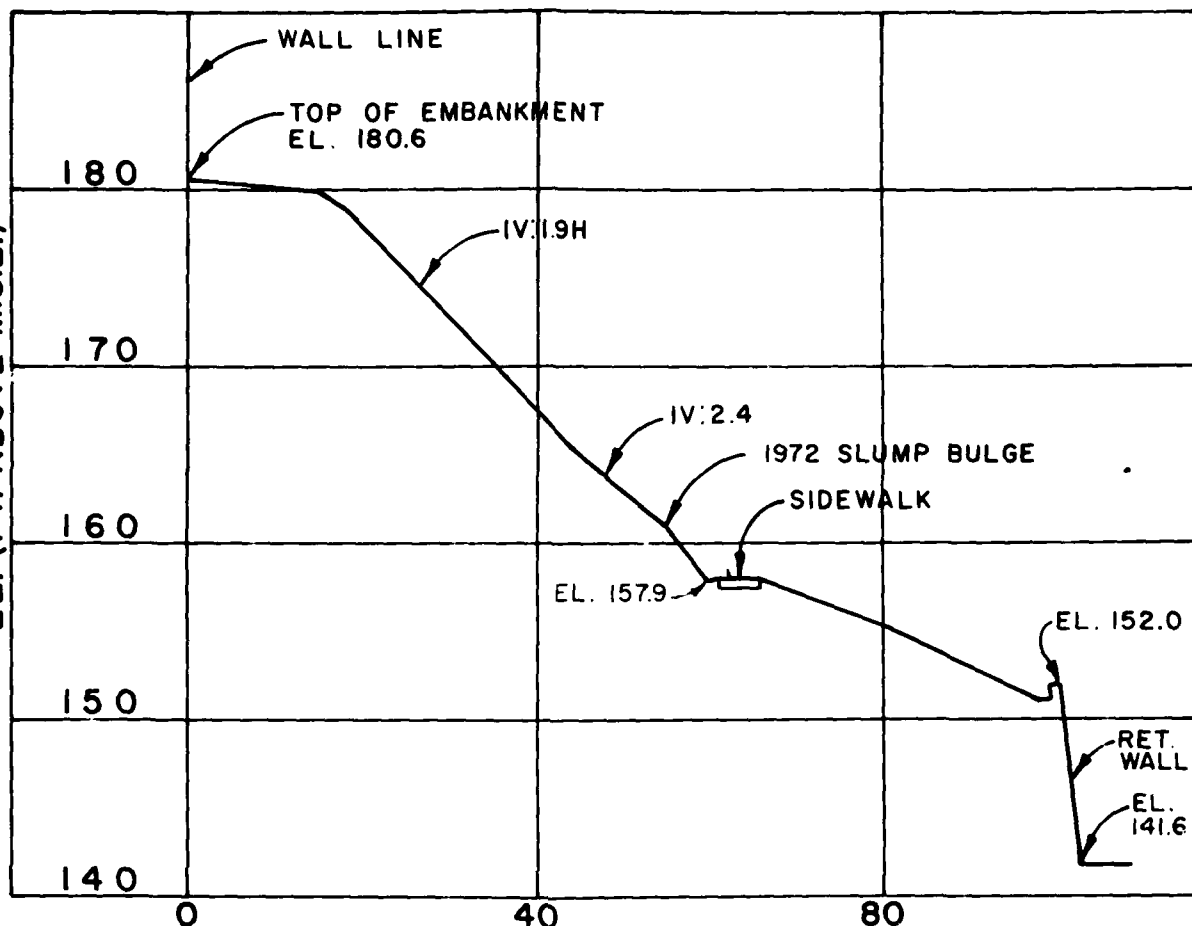
SECTION B (249' EAST OF WEST WALL)

SCALES: 1"=10' V., 1"=20' H.

EL. (FT. ABOVE M.S.L.)

EL. 142.6

EL. (FT. ABOVE M.S.L.)

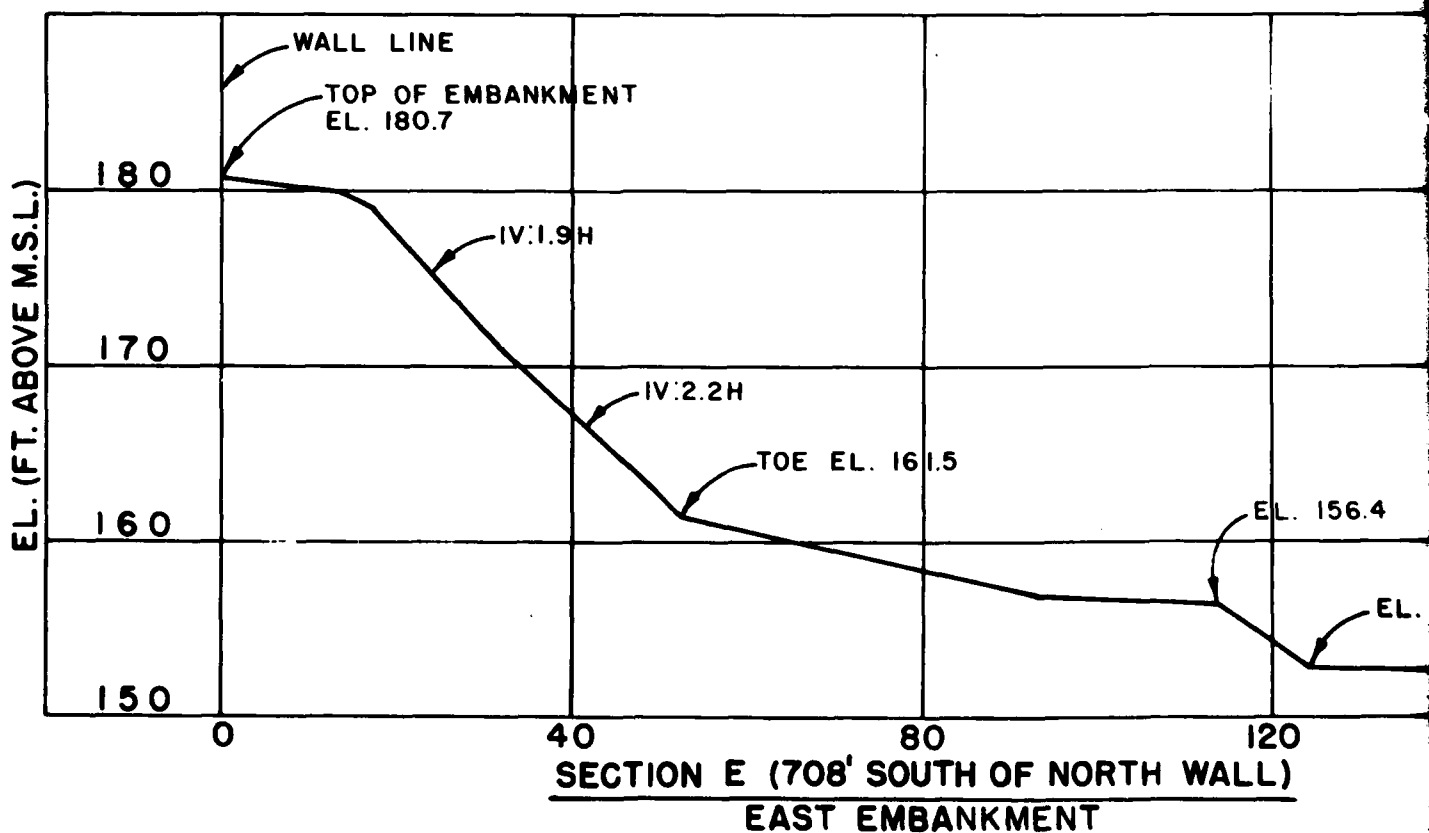
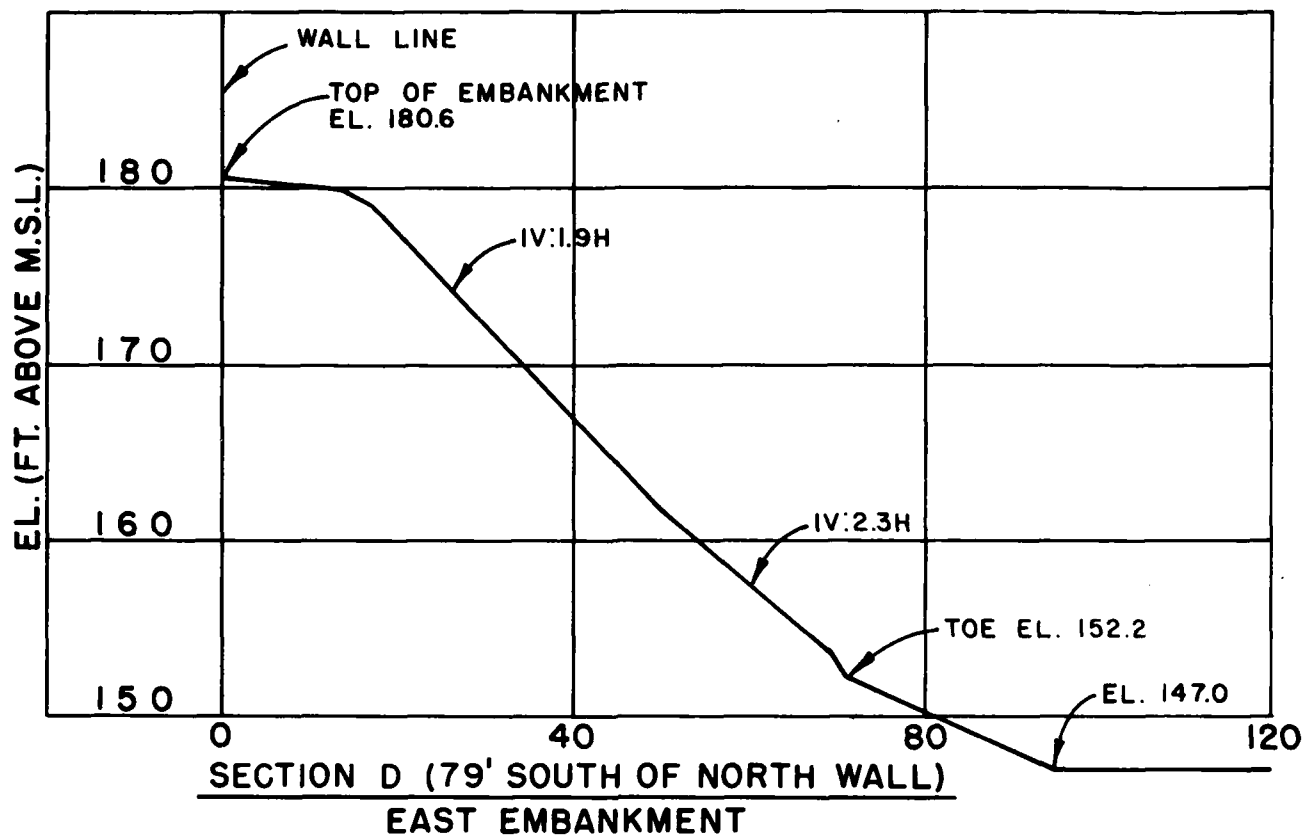


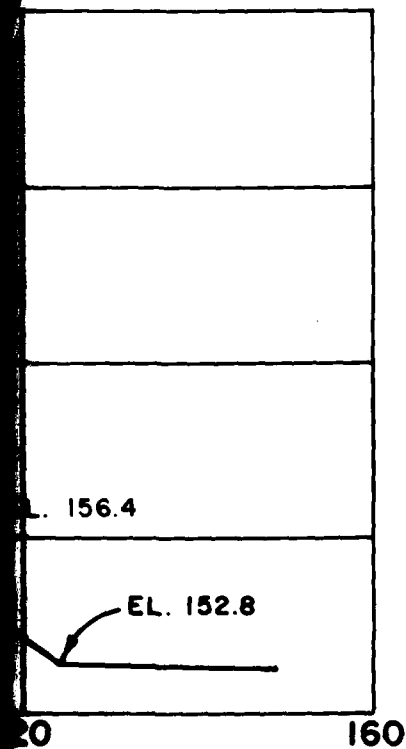
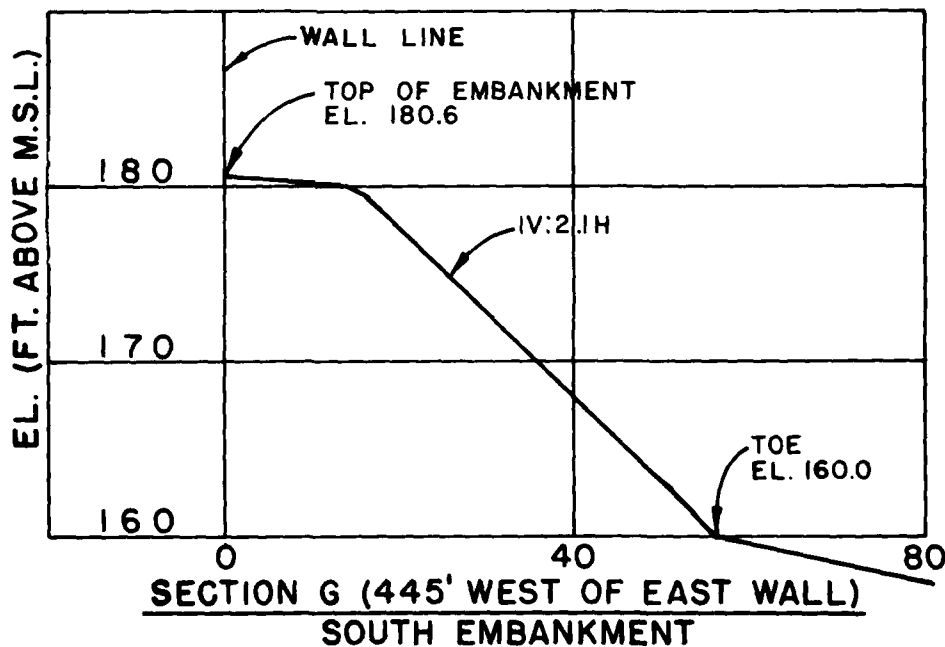
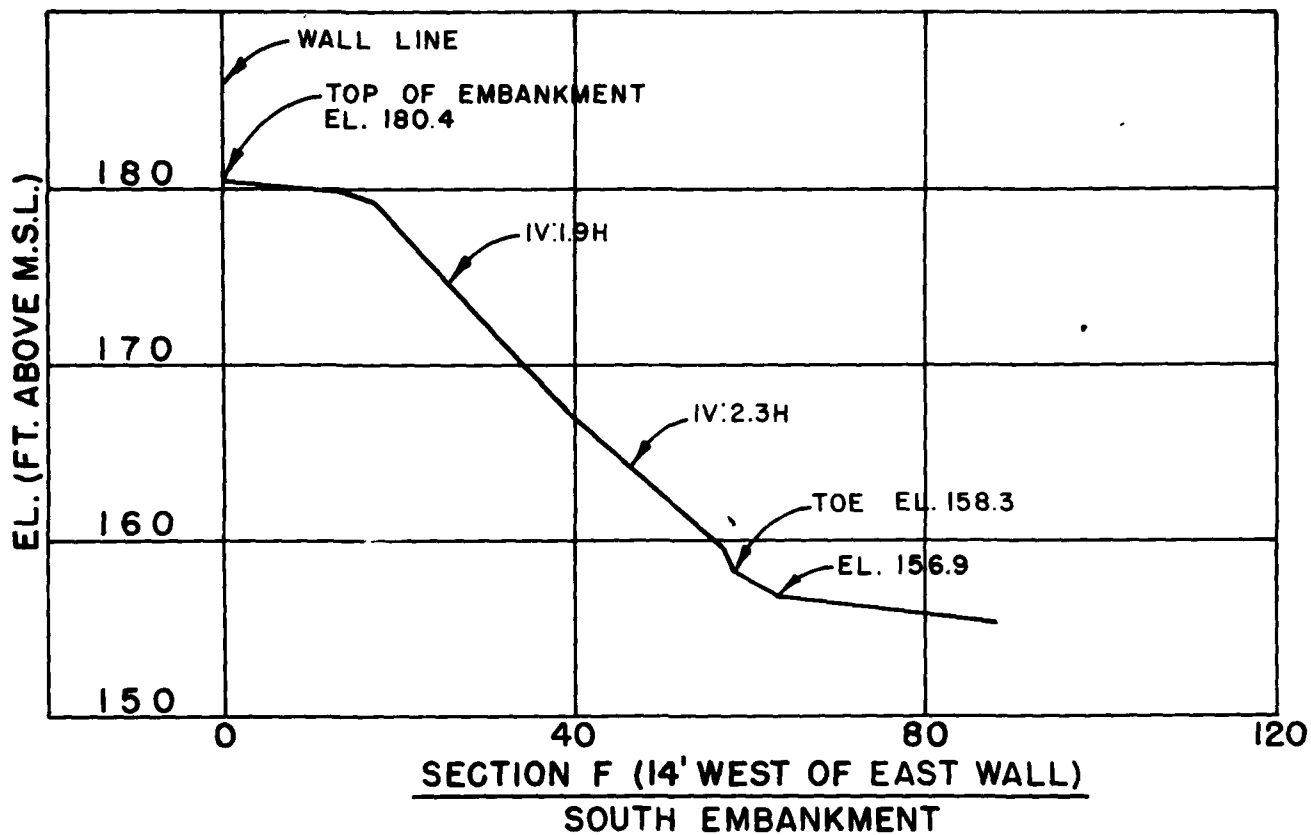
SECTION C (497' EAST OF WEST WALL)

SCALES: 1"=10' V., 1"=20' H.

COMPTON HILL RESERVOIR
CROSS-SECTIONS
NORTH EMBANKMENT

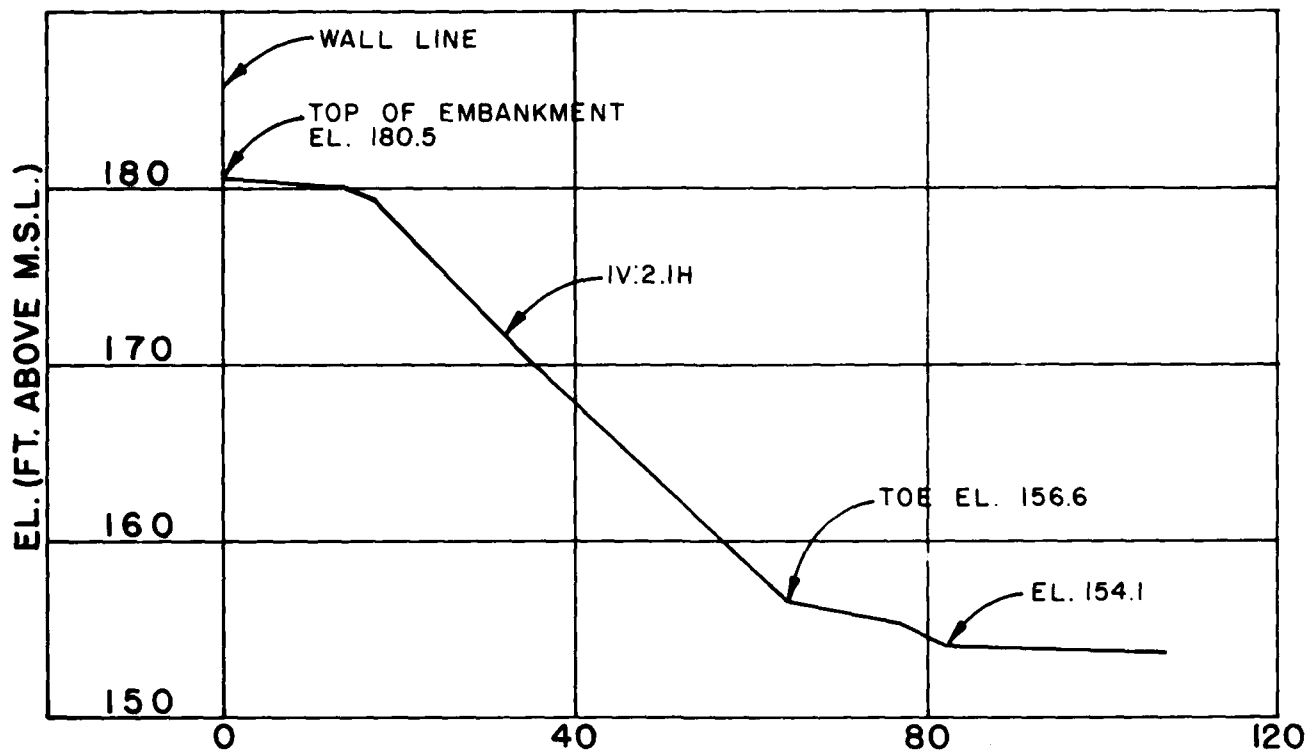
Horner & Shifrin, Inc. March 1981





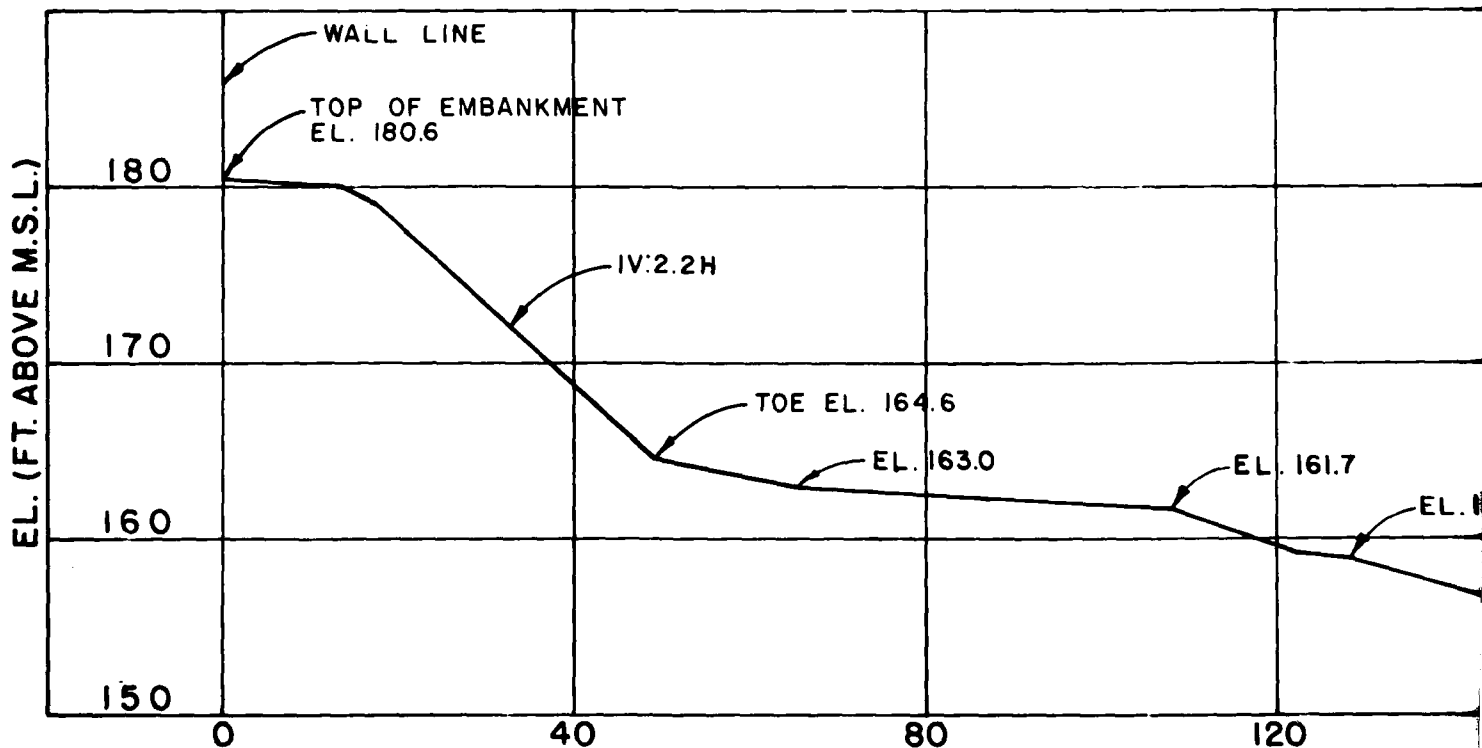
SCALES:
1"=10' V., 1"=20' H.

COMPTON HILL RESERVOIR
CROSS-SECTIONS
SOUTH & EAST EMBANKMENTS
Horner & Shifrin, Inc. March 1981



SECTION H (64' NORTH OF SOUTH WALL)

SCALES: 1"=10' V., 1"=20' H. -

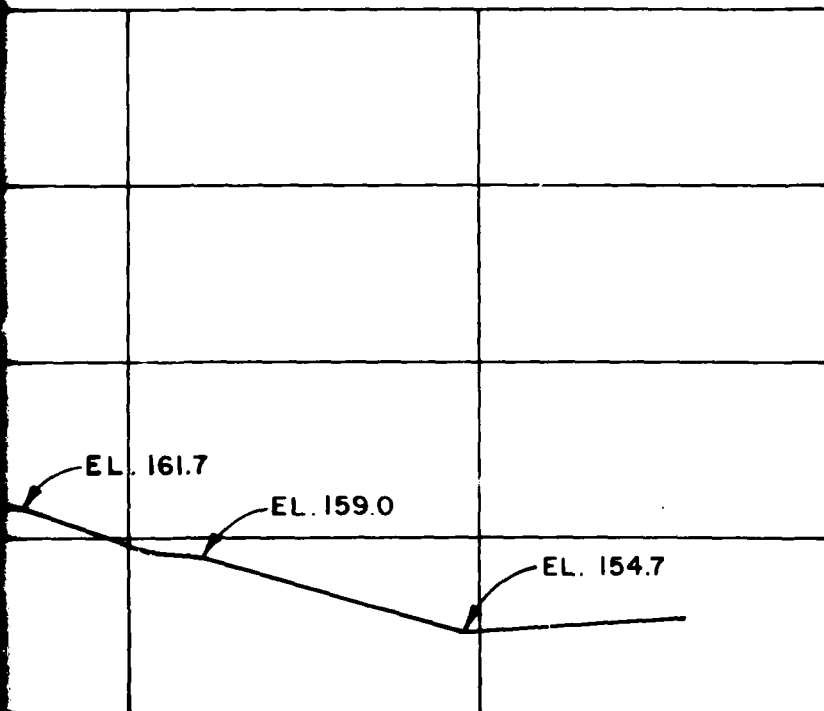


SECTION J (682' NORTH OF SOUTH WALL)

SCALES: 1"=10' V., 1"=20' H.



120



EL. 161.7

EL. 159.0

EL. 154.7

120

160

200

OF SOUTH WALL)

, 1" = 20' H.

COMPTON HILL RESERVOIR
CROSS-SECTIONS
WEST EMBANKMENT

Horner & Shifrin, Inc.

March 1981

AD-A106 204

HORNER AND SHIFRIN INC ST LOUIS MO
NATIONAL DAM SAFETY PROGRAM. COMPTON HILL RESERVOIR (MO 31696);--ETC(U)
JUL 81 R E SAUTHOFF, A B BECKER, G K HASEGAWA DACW43-81-C-0002

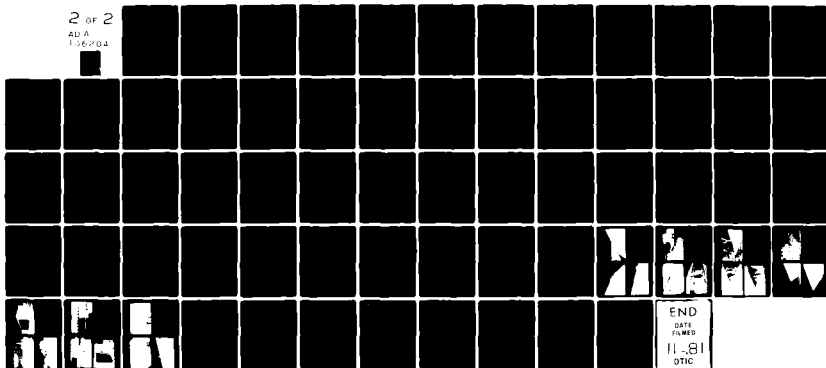
F/G 13/13

NL

UNCLASSIFIED

2 OF 2

AD-A
136204



Excerpt from "Annual Report of the Water Commissioner"
for the Fiscal Year Ending April, 1903

THE WATER COMMISSIONER

135

The results of the tests have not yet been accurately determined, but it is believed that the duty should be about approximately 171,000,000 foot pounds per hour, or about 1000 horsepower.

At the time of the test, the new High Service Engine No. 1 and 2, built especially for the purpose, were being run over a long period with the exception of the engine and the engine proper, a High Service Engine No. 1. The engine and the engine proper, a High Service Engine No. 2, is being run. High Service Engine No. 1 was started on a trial run on March 10th, 1903.

Electric Plant and Railway.

The electric plant at Radon and the electric railway between Radon and the Chain of Rocks have been operated since July during the past year. The cost of furnishing light for the three pumping stations and two water towers, and of operating the electric road was \$9,604 during the year, exclusive of the cost of the fuel or of interest and sinking fund charges. Formerly this service was rendered by private corporations at an annual expense of about \$14,000, of which \$11,000 was for lighting the three stations, and \$5,000 for operating a train for the transportation of the employees of the department. The number of lights has also been materially increased, and the service of the electric car has been much more satisfactory than that formerly rendered by the Burlington railway.

It is to be regretted that the ordinance which authorized the operation of the electric railway for the benefit of the public failed to pass the Municipal Assembly. It would have furnished a new source of revenue for the City, much satisfaction to the people living along the line of the road, and a practical demonstration of municipal ownership, under extremely favorable conditions.

Compton Hill Reservoir.

The repairs on the south basin of the Compton Hill Reservoir were continued during the summer and fall. The faults in the stone paving on the slopes not already so treated, were filled with grout, made of a mixture of about two parts sand to one part of cement in water, and the whole surface was afterwards given five coats of cement wash. Leakage tests which were made on this basin before and after grouting showed a decrease from 150,000 cubic feet to 118,300 cubic feet per 24 hours. These tests were made with the water in the reservoir at an average elevation of 173.7.

Several depressions occurred on the slope near the middle of the east side of the reservoir during January, 1903. Cavities of considerable size, perhaps 10 feet in diameter and 7 or 8 feet deep, suddenly appeared. These depressions were filled with earth and stone. It will be necessary to watch conditions carefully and it is probable that a thorough reconstruction would be advisable at an early date. An examination of the north basin is now being made with a view to making such repairs as will put the basin in permanently safe condition. About \$1,000.00 was expended during the year on repairs of the basins.

The work of improving the grounds in the reservoir park was continued during the year. The roads on the east side of the park were graded and reconstructed and granite gutters were provided. Acting on the recommendation of the Street Commissioner, a new granite walk was laid on the north side of Russell Avenue, from Grand Avenue to Louisiana Avenue, and a granite walk and gutter on the west side of the park. About \$5,800.00 was expended on the grounds and about \$2,500.00 for the granite walk and gutters outside of the park.

Experimental Siphons.

In my last report, reference was made to experimental siphons which were placed between Basins Nos. 1, 2 and 3. These siphons have

SVERDRUP & PARCEL AND ASSOCIATES, Inc.
ENGINEERS - ARCHITECTS
800 N. 12th BOULEVARD
ST. LOUIS, MISSOURI 63101

R. C. WEST
EXECUTIVE VICE PRESIDENT

April 25, 1973

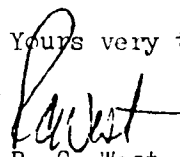
Mr. C. B. Briscoe
Water Commissioner
Department of Public Utilities
City Hall
St. Louis, Missouri 63102

Dear Mr. Briscoe:

Transmitted herewith is a memorandum report of our recent inspection of the Compton Hill Reservoir.

Recommendations are made in the report for continuing repair in preparation for the summer season, and for further inspection. While we do not believe there is the possibility of an immediate or catastrophic failure occurring, there are certain items which need to be examined and taken care of. We will be glad to discuss a continuing program with you at your convenience.

Yours very truly,



R. C. West
Executive Vice President

Enclosure

SVERDRUP & PARCEL AND ASSOCIATES, Inc.
ENGINEERS-ARCHITECTS
600 N. 12TH BOULEVARD
ST. LOUIS, MISSOURI 63101

CITY OF ST. LOUIS
DEPARTMENT OF PUBLIC UTILITIES
WATER DIVISION

A Report on Conditions and Recommendations
concerning the Compton Hill Reservoir

An inspection of the present condition of the North Basin of the Compton Hill Reservoir was made on April 19, 1973. Mr. Stanley Fletcher and others of the St. Louis City Water Department met with Elmer Ott and James Otto of Sverdrup & Parcel and Associates, Inc. to discuss and attempt to determine the problems, the probable causes, and the possible solutions.

The basic situation is that the North Basin of the Compton Hill Reservoir was drained while Highway I-44 was being constructed to the immediate north of the reservoir, and that when the basin was refilled, an objectionable amount of leakage was noticed. The basin was subsequently drained again for inspection and repair.

The reservoir was originally constructed about 1870 with earthen walls, and was reconstructed about 1915 with reinforced concrete walls and floor. The floor system consists of a concrete slab placed over the original puddle clay core, an asphaltic membrane over the slab, and a top wearing slab. About 1930, the roof for the reservoir was installed, utilizing columns which bear on top of the floor system to avoid penetration of the waterproof membrane. After the construction of the roof, the basin remained filled until being drained for the construction of the highway.

Combining a study of the drawings of the structure, a cursory on-site inspection of the interior of the basin, and the reports and comments of the Water Department personnel, it is evident that in addition to the problems of leakage, there are many instances of structural problems in connection with the roof system and its supports. The Water Department desires to refill the basin in time to meet the peak demands of the summer season, and accordingly, our recommended corrective action combines measures to abate the major leakage immediately along with further work to be done after the summer season is past.

The leakage has only been noticed along the north wall of the reservoir, although it is quite probable that some amount of leakage is also present along the other walls. The most apparent cause of the leakage was the drainage of the basin for the construction of the highway. The draw-down was probably very rapid, and the

resulting exterior hydrostatic pressure against the floor forced much of the joint filler material out of the joints, as well as causing many of the floor panels to lift or crack. Further, the duration of the basin being empty allowed the soil beneath the reservoir to dry out to some extent. When the basin was subsequently refilled, the now partially-filled joints allowed a greater leakage which in turn was able to erode through the dried out soil. From the reported location of where the greatest leakage was noticed, it is most likely that the water is flowing out under the slab and wall footing and then over the top of the puddle clay core. Some of the joints in the floor slab which were originally three-quarters of an inch wide are now over 1-1/2 inches wide.

The structural problems noted are due mainly to the fact that the roof slab is approximately 400 feet by 500 feet with the only provision for expansion and contraction being provided along the perimeter of the basin. The forces induced by the movements of the slab have produced flexural stresses in the columns which have caused tension cracks and compressive crushing of the concrete along the outer rows of columns, and this has subsequently allowed corrosion of the reinforcing. The forces have also induced many instances of cracking and spalling of the beams immediately adjacent to the columns. Further, the sliding type expansion joints between the perimeter columns and the roof slab beams are no longer working effectively in many locations. The expansion plates are apparently binding and the columns in many locations have been pushed to the side until the bearing area is reduced to as little as one inch, where originally seven inches had been intended. Further movements of the slab could result in beams being pushed off the columns, which may cause a local collapse of a small portion of the roof cover. In addition to this, there are many beams with hair line cracks as well as many places in which some of the roof slab concrete has spalled away, which in turn exposes the reinforcing to corrosion. The area of concrete within the normal operating levels of the reservoir displays many examples of corrosive attack, due to the immersion and drying cycles. The walls of the basin appeared to be in good condition for a structure almost sixty years old, although considerable spalling has occurred on the exterior of the reservoir.

The open cut for the highway work may possibly have caused some settlement along and under the north wall and slope, but this has not yet been determined. There may have been some rotation of the exterior wall along the north side, or even along the east and west sides, but it would be extremely difficult to establish when this occurred. With the subsequent installation of the highway retaining wall and the backfilling thereof, the lack of any further future settlement cannot be guaranteed, but it should be minimal. A detailed study would be required to analyze what possible effect the open cut had on the reservoir.

In order to use the north basin of the reservoir for this summer's peak demand, our recommendation is for the immediate repair work to be concentrated on repairing the expansion joints in the floor

slab. This should be done principally in the areas of the sloping slabs on the north side of the basin and along the north ends of the east and west sides for a minimum of eighty feet, and for the first bay of the flat portion of the floor immediately adjacent to the sloping slabs. The existing joints should be raked clean to a depth of three inches and a suitable sealant must be installed. The joints over three-quarters of an inch wide should be grouted, as should any cracks in the floor slab. The joint between the vertical wall and the sloping floor slab should be cleaned, including removal of the thin facing on the wall at that point, and then filled with grout. Repair work on the sloping slabs on the east and west sides should be started at the north end and continued as far to the south as time permits. Any repair work to the roof slab or to the columns can be postponed until after the summer season.

While the slab expansion joints are being repaired, the outside of the wall footing in the area of the greatest leakage along the north wall of the reservoir should be excavated and exposed enough to allow visual inspection to see if the earth has fallen away from the bottom of the footing. If it has, then chemical grout should be pumped under the footing to fill the voids. Other areas along the north, as well as along the east and west sides should also be checked and grouted as necessary. These repairs should appreciably reduce, if not eliminate, the excessive leakage from the basin. After refilling the basin, if leakage is still a problem, possibly a diver could be used to locate the problem area with the use of dyes or other means. The ultimate solution to the leakage problem would be the installation of a liner or similar membrane, but this would be very expensive and will probably not be required.

The postponement of any structural repairs until after the summer involves the risk that there may be some partial failures of the roof slab along the perimeter of the basin. These would not cause the failure of the whole roof system or of the walls of the basin, and the area could quickly be covered by a temporary closure. The recommendation to take this acceptable risk is based on a consideration of the time allotted for repairs at this time, as well as the possibility that the load may redistribute and not cause an immediate failure even if the beam does slide beyond the column.

There should be sufficient time available now to make the outlined repairs to the basin to allow its use for this summer. Following the summer demand, when the basin is drained to allow further repair work, it is recommended that the water level be reduced slowly, at the rate of one to two feet per day. This should allow for the exterior hydrostatic pressure to be dissipated to prevent the joint material from being forced out and the possible uplifting of floor slabs. Those joints which were not raked and filled before should then be repaired, concurrently with the repairs which must be made for the structural work.

A comprehensive engineering investigation of the north basin is required to determine what structural repairs are required as well as the most feasible and economical means of accomplishing the repairs. The south basin of the reservoir should also be inspected, utilizing a rubber raft while the basin is reasonably full. Although no leakage problems have been noted, there is definite probability of detecting structural problems and potential failures similar to those found in the North Basin. Any structural problems found should be repaired in a manner similar to that used in the North Basin. These inspections can proceed during the period while the immediate repairs are being made to the north basin, and they should be completed as soon as possible.

Following the inspection of the basins, the preparation of specifications and details for the structural rehabilitation of the roof and its supports should be undertaken. The major problem of expansion and contraction cannot be eliminated, but use of recently developed materials will help insure a system which can cope with the situation. It is apparent that the structure has functioned satisfactorily for the past forty years, and that with a few major improvements and repairs, it will continue to do so. A priority for the repairs can be established to allow the work to be accomplished as time and available funds permit. The highest priority must be allocated to the replacement of expansion bearing plates and the rehabilitation and stabilizing of the columns supporting the perimeter beams. This work must be accomplished at the earliest opportunity. Lower priorities would be set for patching and repairing deteriorated beams, slabs, and columns, based on the individual severity of the problem and on its effect to the integrity of the structure.

The Compton Hills Reservoir is one of the major components of the City Water Department. The utilization of both basins is essential during the summer time to meet the demands of the City of St. Louis. The repairs and recommended work outlined in this report should assure that this structure will continue to serve the community as it has for the past century.

FINDINGS AT
COMPTON HILL RESERVOIR
SPRING, 1973

BY
JOSEPH J. KAMMERER, JR.
CIVIL ENGINEER II
DESIGN AND CONSTRUCTION



CITY OF ST. LOUIS
DEPARTMENT OF PUBLIC UTILITIES
WATER DIVISION

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WALTER T. MALLOY
Director of Public Utilities

August 9, 1973

Mr. S. T. Fletcher
Division Engineer
Design & Construction
City of St. Louis Water Division

Dear Mr. Fletcher:

The accompanying report, entitled "Findings at Compton Hill Reservoir - Spring 1973", is submitted to you as a status report on the project you assigned to me as Work Order #P18158, "Line North Half of Compton Hill Reservoir".

This project began as a search to find a suitable liner for the North Basin of Compton Hill Reservoir. An attempt was made to determine the amount of leakage from the reservoir so as to provide a basis to judge the effectiveness of a new liner. The results of those tests are included in this report.

While investigating the conditions in the North Basin for lining purposes, some areas were found that we felt needed repair. This report contains the method of repairs made in those places. An effort was made to describe other conditions found that need repairs badly, the probable causes of such conditions, comments about them, and the general condition of the reservoir.

Respectfully yours,

Joseph J. Kammerer, Jr.
Civil Engineer II
Design & Construction

JJK/RG/pc

SECTION I

LEAKAGE TESTS

An attempt was made to try to determine the amount of water that is leaking from Compton Hill Reservoir. Since the whole reservoir could not be taken out of service at one time, each basin was isolated at different times, in order to leave the opposite basin in service. The south basin was isolated first. This was accomplished by closing both sluice gates leading to each of the two gate chambers, (four sluice gates in all). The south basin was allowed to sit idle while the north basin continued to function as normal, which in effect cut the reserve supply of water in half.

The elevation of the west hatch frame was determined by survey, to set a reference point. Readings were taken to the hundredth of a foot with a level rod by lowering the rod until the surface of the water was touched and reading the rod at the hatch frame level. Readings were recorded twice daily, usually at 9:30 A.M. and 3:30 P.M.

The south basin was checked by this procedure from February 5, 1973 to February 13, 1973; a total of 8 days. During that time, the level in the south basin dropped from 185.33 to 182.54, a total of 2.79 feet. This calculates out to a total of approximately 4.34 million gallons. That averages out to about 526,000 gallons per day.

However, this figure may be slightly misleading. Looking at the accompanying charts that graph the leakage results and comparing the plot of the elevation of the south basin (isolated) to the plot of the elevation of the north basin (in service) one can see visually that, on the average, the "in service" basin elevation is lower than the "isolated" basin elevation for a greater length of time. This would make no difference if we had a perfect water tight seal between the basins, but we don't.

There was leakage around the sewer gates that was measured in the field to be about 5,000 gallons per day. There are also cracks in the division wall and spaces around the sluice gates that allow water to get from one basin to the other. This happens only when there is a difference in elevation between the two. The head differential will force water through any openings from the high water level to the lower level. This amount will vary with different head differentials and the length of time.

An attempt was made to try to determine what effect these head differences had. The amount of water leaking through is directly proportional to the velocity of the water going through any openings. The velocity is directly proportional to the square root of the head difference, e.g., if the head is doubled, the flow will increase by a factor of $\sqrt{2}$.

Two 18 hour periods were chosen for calculation purposes. These were 3:30 P.M. of February 10, through 9:30 A.M. of February 11, during which period the "in service" basin was at all times below the "isolated" basin, and 3:30 P.M. of February 11, through 9:30 A.M. of February 12, during which period the "in service" basin was at all times above the "isolated" basin. For the first period the basin lost 469,000 gallons in 18 hours or 0.625 MGD. During the second period, the basin lost 202,000 gallons in 18 hours or 0.269 MGD.

Let: A = Actual leak from basin

L_A = leakage between basins when "in service" basin is above "isolated" basin.

L_B = leakage between basins when "in service" basin is below "isolated" basin.

Since the time periods are equal, there is no need to adjust any figures for time. The leakage at the sewer gates will be $\frac{18}{24} \times 5,000$ gallons or 3750. This can be assumed to be 4,000 gallons for convenience.

During the first period when the "in service" basin is lower than the "isolated" basin, the leakage between basins will flow to the "in service" basin, thus increasing the measured amount of leak. An equation can be written to represent this condition as:

$$(1) \quad A + L_B = (469,000 - 4,000) = 465,000 \text{ gallons}$$

During the second period, the leakage between basins will flow to the "isolated" basin, thus decreasing the measured leak. An equation to represent this condition can be written as:

$$(2) \quad A - L_A = (202,000 - 4,000) = 198,000 \text{ gallons}$$

This leaves us two equations in three unknowns. However, L_A and L_B can be related to each other by using the fact that the amount of leakage is directly proportional to the square root of the difference in head. The average head during the first period was found to be 1.15 feet, while during the second period it was found to be 2.40 feet. This means that:

$$\frac{L_A}{L_B} = \frac{\sqrt{h_A}}{\sqrt{h_B}} = \frac{\sqrt{2.40}}{\sqrt{1.15}} = \frac{1.5492}{1.0724} = 1.4446$$

$$\text{therefore: } L_A = 1.4446 L_B$$

By substituting this into our original equations we get:

$$(1) \quad A + L_B = 465,000$$

$$(2) \quad A - 1.4446 L_B = 198,000$$

Subtracting (2) from (1) yields:

$$2.4446 L_B = 267,000$$

$$\text{and } L_B = 109,200 \text{ gallons}$$

$$\text{therefore: } A = 465,000 - L_B = 465,000 - 109,200 \text{ gallons} = 355,800 \text{ gallons}$$

This shows that our actual leakage from the basin during the 18 hours is probably closer to 356,000 gallons than to either of the other figures. This figures out to be about 0.475 MGD.

The same situation holds true for the north basin results. Our drop over 16 days was 1.18 feet, which calculates out to a total loss of 1.834 million gallons or 0.115 MGD. From this we can draw the conclusion that our leakage from the north basin is somewhat less than from the south basin. This can be seen also by noting the plot of the two basins for the two periods, 3:30 P.M. February 24 to 9:30 A.M. February 26, and 3:30 P.M. March 3 to 9:30 A.M. March 5. These two periods are on weekends from Saturday afternoon to Monday morning. During both periods the "in service" basin on the south was at all times higher than the "isolated" basin on the north. This was due to a combination of factors including the cheaper weekend pumping rates which allows us to pump Compton Hill full on weekends, and the very low demand during those two particularly cold weekends. As explained previously, when the "in service" basin is higher than the "isolated" basin, it tends to feed the "isolated" basin, thus decreasing the measured loss of water. In these two instances, the level of the "isolated" basin actually showed a rise. This can only be explained by realizing that in this case, the leakage from the south basin into the north basin must have been greater than the leakage from the north basin into the surrounding soil.

Two time periods were picked for calculation purposes. From 3:30 P.M. of March 3 to 9:30 A.M. of March 5, the "in service" basin was at all times above the "isolated" basin and was therefore feeding it. The level in the isolated basin rose 0.18 feet in 42 hours for a gain of 280,000 gallons. From 10:15 A.M. to 3:30 P.M. of February 21, the "in service" basin was at all times below the "isolated" basin and was therefore taking water from it. In 5.25 hours, the "isolated" basin dropped 0.06 feet, a loss of 93,000 gallons.

The two time periods are not equal so figures had to be adjusted to take that into account. If the basin lost 93,000 gallons in 5-1/4 hours, it would lose 744,000 gallons in 42 hours. During the period of loss, the average head was found to be 3.46 feet. The average head for the period of gain was 2.16 feet.

Let:

A = Actual leakage from basin

L_A = Leakage between basins when "in service" basin is above
"isolated" basin.

L_B = Leakage between basins when "in service" basin is below
"isolated" basin.

Allowing 9,000 gallons leakage from the sewer gate for a 42 hour period, the two equations can be written as:

$$(1) A + L_B = (744,000 - 9,000) = 735,000 \text{ gallons}$$

$$(2) A - L_A = (-280,000 - 9,000) = -289,000 \text{ gallons}$$

L_B can be related to L_A by the equation:

$$\frac{L_B}{L_A} = \frac{\sqrt{3.46}}{\sqrt{2.16}} = \frac{1.86011}{1.46969} = 1.2656$$

therefore: L_B = 1.2656 L_A

Substituting into the original equations give:

$$(1) A + 1.2656 L_A = 735,000 \text{ gallons}$$

$$(2) A - L_A = -289,000 \text{ gallons}$$

By subtracting (2) from (1), the equations yields:

$$2.2656 L_A = 1,024,000$$

$$L_A = 452,000$$

therefore: A = 452,000 - 289,000 = 163,000 gallons

The 163,000 gallons loss is spread over 42 hours so it averages out to 0.093 MGD. Just as the calculations for the south basin showed, the calculated actual leak of the north basin, 0.093 MGD, is slightly less than the

average measured leak, 0.115 MGD. This is because the "in service" basin is below the "isolated" basin for slightly longer periods of time, during which periods the magnitude of the average head differential is greater also.

In summary, it is concluded that the south basin is leaking about 0.475 MGD and the north basin is leaking about 0.093 MGD. The total leakage into the surrounding soil is 0.568 MGD. If leakage from the sewer gates is included, this becomes 0.578 MGD.

The condition found in the field is quite different than anyone imagined. It was expected that the north basin would leak much worse than the south basin, but quite the opposite is true. The south appears to leak about 4 to 5 times as much as the north.

Apparently, the water leaking out of the south basin has a natural underground drainage system that allows the water to dissipate in the surrounding park soil without causing any damage. The water leaking out of the north basin seemingly has no such system, because it is surfacing on the north slope and causing some erosion problems.

Both basins have been suspected of leaking for quite some time. The amount of leakage out of the north basin is quite a bit less than anticipated though. I would conclude therefore that it cannot be any worse than it had ever been for the last ten to twenty years because it is a surprisingly minimal amount now. In view of the fact that the south basin is leaking 5 times as much as the north basin and is causing no damage to any surrounding slopes, and since there is no apparent recent increase in the magnitude of the leak from the north basin, it seems obvious to me that the new highway, I-44, has caused our erosion problem by disturbing the natural underground drainage system that we must have always had, and reducing greatly the drainage area near the north slope.

SECTION II

STRUCTURAL CONDITIONS FOUND AND REPAIRS MADE

Nothing much can be reported about the south basin. It was drained for only a few hours, which gave time for only a very cursory inspection at best. The flashlights used were standard Water Division issue, which were not strong enough or bright enough to see to the top of the slope at the walls.

There was not time enough to climb any slopes, just time enough for a quick walk through the basin. It was made clear before the draining that no repairs of any kind would be made to the south basin unless some severely drastic situation was found that needed emergency attention. Therefore, the south basin was inspected with the purpose of looking only for some kind of major disaster. None was found.

The north basin, on the other hand, was inspected quite extensively. Quite a few measurements were taken so as to provide a comparison for future measurements.

The floor and slopes of the reservoir and the roof were cross-sectioned by survey. In this way, we can check if any settling is going on. The floor of the reservoir can be checked only when the reservoir is drained. However, if the area of any possible settlement includes a column, the column must sink along with the floor. The roof will then sink with the column and this settlement can be located and determined by survey of the roof. The advantage of the roof survey is that it can be done anytime, without the need to drain the reservoir.

In two places along the west side of the reservoir at the foot of the side slope, we found uplift failure of the floor slabs. Slabs were pushed up along joints to a maximum difference of 4 inches between adjacent slabs.

Most of these slabs were also cracked. They appeared to be quite fresh when first seen because the sides of the uplifted slabs and the cracks were clean. Over the rest of the reservoir's interior surface was a thin layer of calcium carbonate precipitate which covered absolutely everything. The absolute cleanliness of the slab sides and cracks showed they were very fresh.

The uplift had to be caused by external hydrostatic pressure. The reservoir floor is normally under 20-32 feet of water. Leakage from the reservoir apparently builds up to equalize the pressure of the head of water. When the reservoir was drained quickly, the ground water could not dissipate quickly enough outside, or back into the reservoir. With no water inside the reservoir to neutralize the external hydrostatic pressure, some floor slabs were pushed up and cracked in the process. Apparently this was enough to act as a pressure release mechanism because only two such places were found.

The same draining procedure was used this time as was used in the fall of 1970. The sewer gate was opened all the way and the water was allowed to drain out as fast as the 20 inch sewer could carry it away. This takes 36-48 hours.

An interesting question can be asked at this point. If the reservoir is not leaking any more now than in 1970, (which is a conclusion drawn previously in the leakage discussion), and if it was drained in the same manner, very quickly, then why did we not encounter uplift problems three years ago?

I believe the answer to be that the new highway, I-44, has disturbed the underground drainage and does not allow the leakage water to drain away as it once did. This in effect, dams up the water which collects in the soil surrounding the reservoir and builds up hydrostatic pressure equal to the head of water in the reservoir. When the reservoir is drained quickly, the hydrostatic pressure cannot dissipate quick enough and the resulting difference in pressure causes uplift.

It was recommended by Sverdrup and Parcel in their report on Compton Hill that the reservoir should be drained at the rate of 1 to 2 feet per day. After filling the north basin for the purpose of chlorination, we drained the basin at a slower rate. Since the basin level fluctuates commonly between elevations 175 and 186 daily, it was felt that we needed to drain slowly only below that level. It was drained quickly down to about 175 and then slowed. We felt we could do the remaining 21 feet at a rate of 4 feet per day. This was done.

We were allowed a few hours to inspect it between that draining and the filling of it to put it back in service. We found one place near the center of the north wall at the foot of the slope where uplift occurred again. Apparently 4 feet per day is also too fast and next time, it should be drained at the recommended 1 to 2 feet per day.

The uplift found after chlorination was not repaired in any way because the basin was urgently needed to be put back into service for the summer. The other two places found after the first draining were repaired.

At both locations, shown on Drawing #4532, the 3 inch upper slab was broken out and hauled away. The water proofing membrane was scraped off the lower slab and thrown away also. In one spot the lower slab was also cracked and a 3 foot by 1 foot section was removed.

The base beneath the lower slab of concrete is macadam. It was found to be moist but seemed very solid and stable still. The small hole in the lower slab was then filled with concrete.

We put down a new waterproofing membrane to replace the other that was removed. Materials used were polyethalene sheeting, asphalt impregnated cotton fabric, and a tar based sealant. The sealant was spread over the cotton fabric with trowels. Four layers each of the cotton fabric and sealant were put down to form the membrane. A single sheet of polyethalene

sheeting was put down on top of that to insure that the fresh concrete poured above would not mix with the sealant in any way.

New concrete slabs were poured over this membrane and new expansion joints formed. This was done by using boards of the desired width as forms for the slab. Before the concrete was completely set, the boards were pulled out leaving openings between slabs. After the concrete was set, ethafoam rods were forced into these joints as a filler material. A waterproof expansion joint material that bonds to concrete was applied with a caulking gun to complete the slab repair.

The division wall appears to be in sound structural condition. There was water leaking through several expansion joints and cracks in the wall. While they did allow water to pass through, the cracks do not appear to effect its structural capabilities. When the weather got warmer and the water in the other basin got warmer, the wall, through thermal expansion, began to squeeze the cracks and expansion joints tighter and the flow of water leaking through slowed noticeably.

The sluice gates in the two gate chambers leak a little bit, also, but they do as good a job as can be expected, considering their age and infrequency of use. The sewer gate leaks pretty badly when the basin is filled and has to be caulked before the filling to hold this leakage to a minimum. The stem is slightly bent from the effort of trying to get it to close tightly.

All together, between the cracks in the wall, the expansion joints, and the sluice gates; there is quite a bit of water constantly flowing into the basin. The only openings, when drained, are two manholes; the sewer gate; and an overflow drain. As a consequence there is very little air circulation in the basin and it is always 100% humidity in there. Moisture is constantly collecting on the ceiling and then falling down in droplet form. The

basin would be very difficult to dry out unless additional holes were cut into the roof to allow forced air circulation with large fans.

The slopes were all in good shape. In some places, though, the joint material did not appear to be in good shape. In others, it looked like the pitch material forming the waterproofing membrane had been squeezed out through the joints. It gave an appearance of having flowed out of the joints.

The first expansion joint south of the northeast corner running up and down the east slope had shown separation. The asphalt expansion joint material was still in the joint but the slabs had pulled apart about 3/4 inch. This was the only joint found that showed separation. The old joint material was cleaned out and the gap was stuffed with ethafoam rods. The same sealant was used here as was used in the floor slab replacement.

The north slope was gone over very carefully, along with the first 120 feet of the east slope south of the northeast corner. All joints that did not look good were marked. These joints were chipped out with chipping hammers to make a clean surface for new joints. Asphalt expansion joints were placed between slabs and the rest of the chipped out spaces were filled in with Embeco grout.

In some places, there were openings between the slopes and the vertical wall at the point where the two meet. As a general rule, these gaps were bigger near the northeast corner and only hairline everywhere else. These gaps usually occurred only on slope slabs that had columns sitting on them. Adjacent slope slabs that did not have columns sitting on them exhibited only a hairline crack at worst and usually nothing at all. It appeared that the slope slabs were moving away from the vertical wall and not vice-versa.

The entire north wall and north 120 feet of the east wall were checked. Where the gap appeared worse than just hairline it was chipped out and filled with ethafoam rods. The same sealant used around the floor slab

repairs was used in this gap as well.

The vertical wall is still in good shape structurally. It needs no work done on it on the inside.

It is spalling badly on the outside though. The spalling occurs because the concrete wall absorbs moisture from the basin. The walls are constantly saturated, and the freezing-thawing cycle causes the spalling. The walls were worked on by applying a coating of gunnite in 1960. However, it is not the gunnite that is spalling off now. Thin layers of the original concrete beneath the gunnite are spalling off and the gunnite, still holding tight, falls with it.

The roof appears to be sound and should be alright as long as its not overloaded. It should not be loaded more than 30 PSF.

The outside surface of the north basin roof had been topped with about 1-1/2 inches of mortar in the late 1940's. It is in very good condition with very little spalling in evidence.

The inside surface shows some scattered spalling. In these places there are reinforcing bars exposed. The roof beams in general looked good but they could not be inspected too closely. We did not have the opportunity or time to get a ladder in for such close inspection. As a result, we looked at most of them using a flashlight from the floor 35 feet below. The ones near the top of the slope at the edges of the roof allowed us a closer look. There were a few of these that showed cracks but that could be attributed directly to other causes. The others that we could see were in good shape.

The most obvious thing wrong in the basin were the columns around the edge of the roof. Since these columns are built at the top of the slope, they are much shorter than any other columns. They are 7-1/2 feet high at the wall and either 9-1/4 feet high or 12 feet high at the end furthest

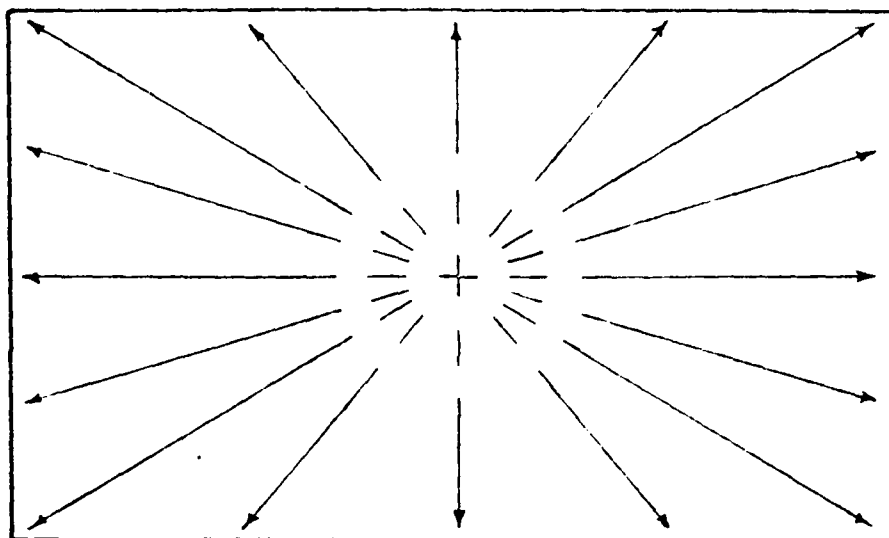
down the slope, (9-1/4 feet on the east and west sides and 12 feet on the north side). Because the columns were poured abutting to the vertical wall, allowances had to be made for expansion and contraction of the roof. Other columns are tied in structurally with the roof and these columns are free to move with the roof at the top. However, the columns at the edge are restricted by the vertical wall, thus expansion and contraction had to be taken into account.

A 12 inch wide brass plate was cast into the top of each column. A 9 inch wide brass plate was cast into the bottom of the roof beams where they rested on the columns. The two brass plates in contact are supposed to slide upon one another and allow enough small movement to take care of the thermal expansion of the roof.

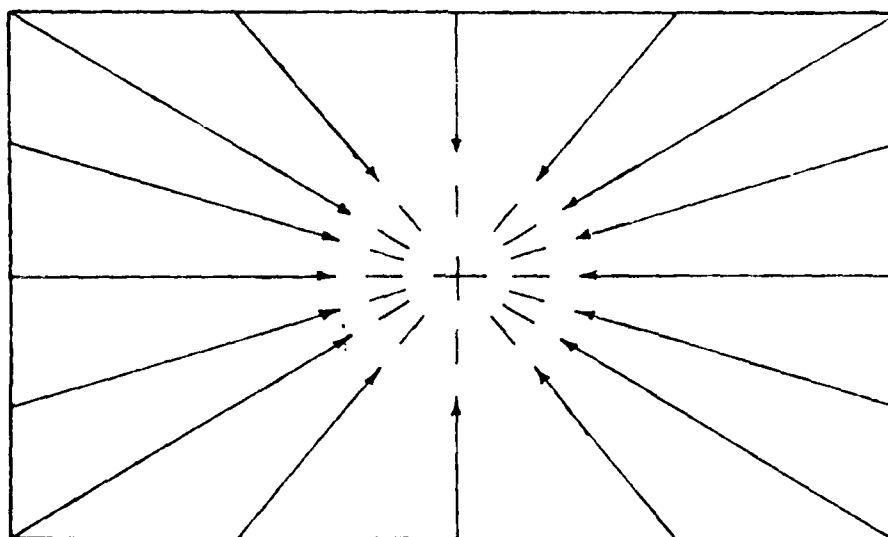
As seen in Figure 1, the roof expands and contracts in all directions from the center, (although it is not shown, it expands and contracts in thickness, also). The coefficient of thermal expansion of concrete is 6×10^{-6} per degree F. Therefore, at 95° F in the summer, our 500 foot long roof is 3.6 inches longer than it is at -5° F in the winter. At a column near the extreme edge of the roof, such as Column D in Figure 2, the roof and its beams under these conditions would move 1.8 inches on the supporting column between its largest and smallest size.

Figure 1 shows the directions of the horizontal forces that the roof exerts on the columns during expansion and contraction. Figure 2 shows these forces broken down into free body diagrams for three various positions. At position A, the line of force is at 45° to the wall; at B, 60°; and at C, it is perpendicular to the wall. At A, the component of the force perpendicular to the wall is equal to the component of the force parallel to the wall. As the force approaches perpendicular, as in B, the perpendicular component of the force increases and the parallel component decreases. When the force is perpendicular, as in C, there is no parallel component and there is no force on the column to either side.

Chart 4-21

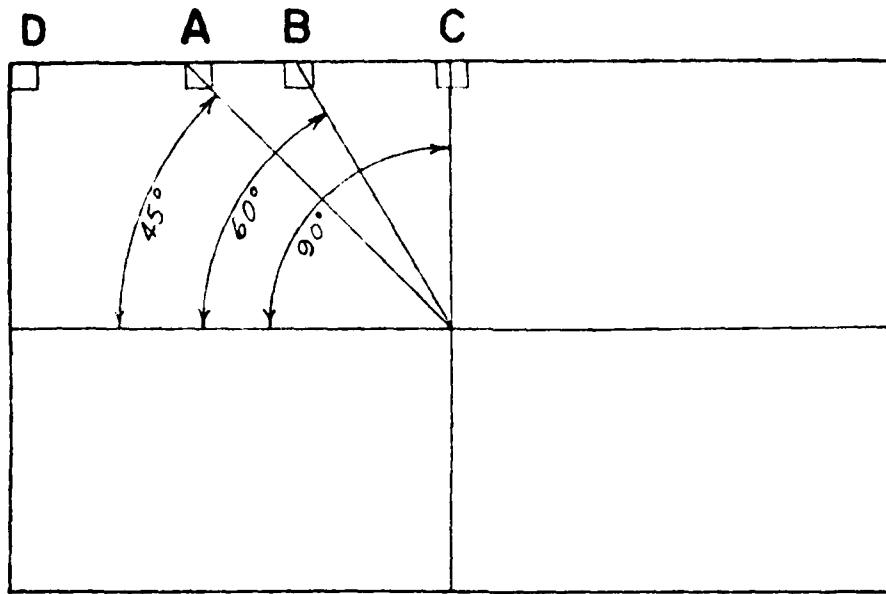


DIRECTION OF EXPANSION

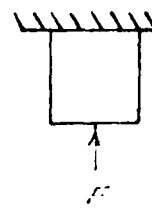
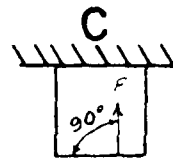
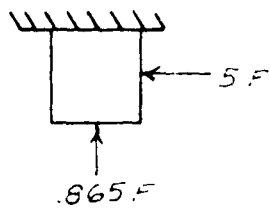
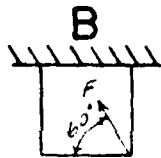
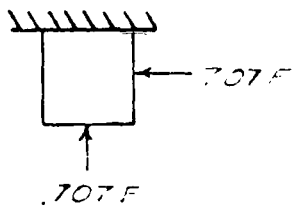
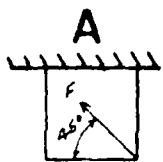


DIRECTION OF CONTRACTION

FIGURE 1



DURING EXPANSION



DURING CONTRACTION

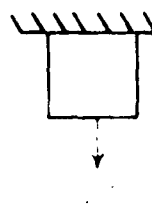
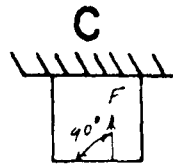
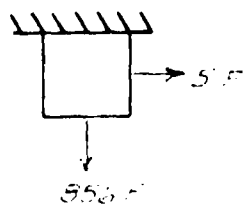
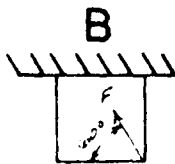
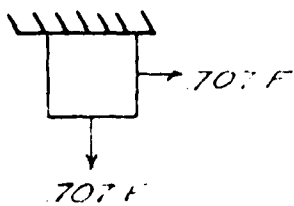
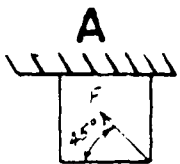


FIGURE 2

The total displacement or movement by the slab is greatest at the corners. That is because the diagonal distance to the corner is greater than any other. Such movements are not on a seasonal basis from summer to winter. They occur daily. As the temperature rises or drops even a few degrees, the roof slab is expanding or contracting.

In our particular case the brass plates are supposed to allow the expansion and contraction movement to take place. The columns are supposed to resist movement by being connected to concrete pads that run up the entire length of the slope. These pads resist the moment caused by the application of forces at the top of the column by the roof. If everything is working right, this amount of resistance is greater than the friction forces between the two brass plates and the brass plates then slide upon each other.

The conditions found in Compton Hill indicate that our brass plates are no longer working as well as planned. Columns around the edges are leaning over to various degrees. As explained previously, the columns at the extreme edge receive the most lateral force. The columns near the center, the ones closest to the perpendicular, receive very little lateral force. As can then be expected, the columns nearest the corners lean the most. In some cases the column appears to be as much as 7 inches out of plumb. Near the center of the walls, the columns still appear to be very close to plumb. The further from the center of the wall the column is, the further out of plumb it is.

The columns do not lean out of plumb on a haphazard pattern. They all lean toward the center of the wall. This tells us that the brass plates slide alright while the roof is expanding but bind up during the contraction cycle and the contraction forces pull all the columns toward the center.

In all probability this has been going on constantly since the roof was put on. However, the condition seems to be worsening rapidly. When the

reservoir was drained in the fall of 1970, it was inspected then. All inspection work was done by Water Division personnel. I talked to two workers and one supervisor who had been there in the fall of 1970. Both workers saw the present conditions and the supervisor was shown slides of our present conditions. All three had climbed the slopes to inspect both corners, where the columns at present are furthest out of plumb. All three men told me that they noticed no columns out of plumb at that time and all three tell me that they would have noticed the columns leaning if the columns would have been as far out of plumb then as they are now.

The fact that none of the three men noticed the columns out of plumb leads me to believe that most of this movement has taken place in the last two and a half years. The columns may have been just beginning to go out of line then and no one would have noticed that. However, once something like this starts, it gets progressively worse. The more the column leans, the more the brass plates tend to bind up, and the column will be moved an even greater amount on each successive cycle.

At this point I began to wonder if there was any relationship between our problem and the construction of I-44. It seemed strange that all this movement would occur since our draining the reservoir 2-1/2 years ago for the highway construction. As yet, I have found no correlation between the two facts. Since all the damage can be directly attributed to expansion and contraction of the roof, it appears to be just a coincidence that they occurred together.

There may be some structural trouble if this is allowed to continue as is. Some of these columns are not far away from leaning so far as to give no support to ends of some roof beams. If this happens, there may be some local failure of the roof slab in that location. This situation has been pointed out before in the report on Compton Hill by Sverdrup and Parcel.

The soils experts at Sverdrup and Parcel tell us that any settlement along the north slope due to the highway construction should be finished and any more settlement will be very minimal. With that fact taken into consideration, I agree with the Sverdrup and Parcel report that correcting our problem with the columns should have highest priority. These columns must be plumbed and repaired. To avoid the brass plates binding again, some substance should be found that can act as a greaseless lubricant between the plates, for example, a substance like Teflon.

Other columns in the basin also show the effects of expansion and contraction. All columns, except for a very few near the center, show tension cracks. These are caused by the expansion cycle. Every column, except the previously discussed perimeter columns, are tied in structurally with the roof slab. Since the two basins are covered by two individual roof slabs, the center around which the expansion and contraction take place lies directly over the center of the basin.

As the roof slab expands, the top of each column is pushed in an outward direction away from the center, just as the arrows in the expansion diagram of Figure 1 point. The bottom of each column is held firm and does not move. This has an effect of bending the columns away from the center.

Because of the effects of thermal expansion, the greater the distance from the center the column is, the further the top of the column is pushed. Near the center, the columns are bent very little and no tension cracks show on any columns. However, only a short distance from the center, the columns are bent enough to cause tension cracks in their sides. Concrete is very weak in tension and logically tension cracks would be much more prevalent than compression failures.

All tension cracks appear on the inside edge of the column, that is, on the edge nearest to the center of the basin. At the columns furthest from

the center, near the NE and NW corners, some localized compression failures have been found, also. The columns have not failed; just some small areas of the column show a flakey, rose petal type configuration typical of crushed concrete. This crushing is found only near the corners on the short columns half way up the slope, and are on the side of the column away from the center of the basin. The longer columns at the floor level can take the same amount of movement without crushing because of their much greater length.

Most of these tension cracks appear to be only surface cracks that harm nothing. However, many are deep enough, 1-1/2 inches or more, that the reinforcing bars are rusting out. Reinforcing bars are also rusting out of some of the struts connecting the columns. The rusting is not very extensive and is not critical at present.

At this time, the structural repair has been given priority over lining the structure. This is the only way to do it because there is no lining product on the market that will hold the reservoir together if a failure should occur. It is best to do the structural repairs now and line it later. Before lining takes place, the reservoir should be grouted to fill possible voids beneath the structure and guard against any possible settlement problems.

Lining the reservoir will undoubtedly stop any leakage problem we have. It is, however, very costly. There is one beneficial side effect worth mentioning that would result from the lining job that would save us money in another area involving maintenance. That is the problem of spalling on the outside walls. As explained before, the spalling is the result of water in the outside layers of the saturated concrete walls freezing and popping the concrete off. This will continue to happen as long as the walls are in their saturated state.

If the reservoir were lined, the walls would eventually dry out and only normal weathering would take place, as in any ordinary concrete structure. The wall would have to be restored one more time, after a period of time to allow the wall to dry out.

The walls of Compton Hill are not in good shape now, and will need restoration in the near future. The last restoration job was completed in October of 1960 at a cost of \$123,673.95, only 13 years ago. The walls would have to be restored one more time after lining, but after that, the spalling would cease.

One thing should be pointed out and that is that present plans are to line the north basin. This would stop spalling on the outside of the north basin only and would not affect the spalling on the south basin. To stop the spalling everywhere, both basins would have to be lined.

When the opportunity occurs, the south basin should be examined more closely. The same expansion and contraction that has caused us structural problems in the north half, has been occurring in the roof slab over the south basin also. It's possible that the brass plates have not been binding and have not pulled any columns over, but its just as possible that they have. On our cursory inspection of the south basin this spring we did not have the time or equipment to check this, even had we known enough to look for it. I cannot say at this time what condition the perimeter columns of the south basin are in. Therefore, they should also be checked as soon as possible.

SECTION III
FOOTING EXCAVATIONS

A series of excavations were made to check the condition of the footing beneath the vertical wall. Their sizes and locations are shown on Drawings #4532 and #4533.

The first four holes were dug while the north basin was still drained. They were located along the north wall at distances from the northeast corner of 40 feet, 130 feet, 240 feet, and 350 feet. We found a space between the footing and the soil beneath it. Apparently, the dirt has settled away from the footing. The space was 1-1/2 inches in #1 and lessened gradually in the four holes to about 1/2 inch in #4.

Excavation #1 and #2 were tunneled back to the steel sheetpiling, which is about 4 feet back from the edge of the footing. The steel sheetpile was cast into the footing at a location of approximately 1 to 1-1/2 feet outside of the exterior surface of the vertical wall. Only the top 2-3 feet of the steel sheetpile was exposed. It seemed in good shape. The footing in excavation #1 had a vertical crack in it and one concrete pile had a tension crack in it. Both were small.

Because of the cracks found in excavation #1, we dug two more holes. Both were 10 feet from the northeast corner on the north and east walls. They were also tunneled back to the steel sheetpile. No cracks were found in any concrete piles or the footing, but a small hole was found in the steel sheetpile in excavation #5.

All these excavations were left open until the basin was filled. The filling took place over a weekend and when we returned that Monday morning, all the excavations had water in them. The water level was half way up the footing, approximately at 177-178.

We tried to pump them out but water kept flowing into the excavations

on the north side from the east. We began to use a second pump. It was placed in the excavation on the east wall and the other pump was used on the excavations on the north wall. Water was observed entering excavation #6 from the south side along the underside of the footing next to the steel sheetpile. While the pump in hole #6 was operated constantly, the other excavations remained void of any water or water flow. The flow out of the pump in hole #6 was estimated at 55,000 gallons per day.

No water was observed flowing out of the hole in the steel sheetpile in hole #5, so excavation #1-#5 were backfilled. Excavation #6 was kept pumped out during working hours, but filled again every evening.

We decided to investigate where the flow was originating, if possible. To that end, excavation #7, #8, and #9 were dug. These were each backfilled in turn before digging the next one because the water flowing next to the steel sheetpile just beneath the footing was each time observed to flow from the north.

The excavation labeled #10 was actually just an enlargement of #6. We found the source of the water in this one. It was an approximately round hole roughly 2 inches in diameter located 17-1/2 feet from the exterior surface of the vertical wall of the northeast corner. It was about 6 inches below the footing. An estimated 95% of the flow was coming from this hole while the other 5% came from the south, (the same as was noted in holes #7-#9).

We kept this hole pumped out by running a small gasoline pump around the clock for two days. The flow was directed down the steps at the northeast corner and flowing away from the north slope. During this time, the north slope dried noticeably and the flow of water across the sidewalk was stopped.

We concluded that if we stopped the flow of water out of the sheetpile that we would have no more problems with the north slope, and could save the money budgeted for the lining. It was felt that piping this water away to the nearest sewer could lead to bigger problems. The water could carry away small particles of soil from underneath the reservoir and cause unwanted settlement or collapse.

It was decided to try to plug the hole. A wooden plug was driven into the hole and a quick setting mortar was applied around the plug. When checked the following morning, the excavation had water in it. After pumping it out, it was observed that the plug was holding and was watertight. There was a flow of water coming into the excavation from the north side, where previously there was none.

We re-excavated hole #5 to check to see if the water had taken a slight change in path and was using the hole in the sheetpile observed earlier in #5. It was not. No water was coming through the sheetpile at that point. We then plugged the hole with quick setting mortar anyway. Water was flowing along the sheetpile beneath the footing and was coming from the west, where previously there was no flow. We then backfilled #5.

We decided not to pursue locating the source of water any further. We felt that no matter what we did, the water would find another way out.

As long as we had the equipment on the job, we excavated in other locations to compare them to our trouble spot at the northeast corner.

Excavation #11 was dug 30 feet north of the division wall. A gap was found under the footing that measured 1-1/2 inches. No water was observed flowing anywhere, but we did not tunnel back to the sheetpile.

Excavation #12 was dug 30 feet south of the division wall. As the hole was dug below the footing, soil began sloughing away from under the footing and water was seen coming into the hole. Further digging under the footing

exposed the end of a very rusty 6 inch water main. Water was trickling out of the end of the pipe and also out of the pipe trench surrounding the pipe. The pipe was running parallel to the vertical wall, under the footing in a northerly direction towards the east gate chamber. No further exploratory work was done.

Excavation #13 was dug at the east wall, 10 feet north of the southeast corner. A gap of 1-1/2 inches was observed beneath the edge of the footing, but no water was seen. The hole was not tunneled back to the steel sheetpile. Excavation #13 was backfilled after observation was completed as was #11 and #12. We felt that further digging would not produce any new information so excavation investigation was halted.

UNITED STATES DEPARTMENT OF COMMERCE

NAVAL ARCHITECTURE

BY
Stephen J. Kende
Civil Engineer II
Marine and Construction

Chart 4-32



CITY OF ST. LOUIS
DEPARTMENT OF PUBLIC UTILITIES
WATER DIVISION

DONALD C. GUILLEY, COMMISSIONER
1610 SO. KINGS HIGHWAY
ST. LOUIS, MISSOURI-63110

JOHN H. POELKER
Mayor

WALTER T. MALLOY
Director of Public Utilities

STANLEY T. FLETCHER

ARLE E. THOMASCHER
SUPERVISOR OF DESIGN

GEORGE H. FLETCHER
SUPERVISOR

STANLEY T. FLETCHER
DESIGN & CONSTRUCTION
227-488

HAROLD B. WELGE
SUPERVISOR
227-462

June 28, 1977

Mr. Michael Brynac
Division Engineer
Design & Construction
City of St. Louis Water Division

Dear Mr. Brynac:

The accompanying report entitled, "Slope Stability Study of Compton Hill Reservoir" is submitted to you as verbally requested by Stanley T. Fletcher in the Spring of 1975.

This project stemmed from the fact that a slight failure occurred on the north slope of the reservoir due to leakage. A French Drain was then installed to solve the problem. The objective of this study was to determine the effectiveness of the drain and the present condition of the north slope.

Included in this report are the methods by which data was obtained and the analysis of that data. Because of the irregularity of the information obtained measurements were taken over a two year period in order to form a more definite pattern.

I wish, at this time, to thank the employees of the Supply & Verifying, Design & Construction, and Operating Sections for their cooperation and help in obtaining and analyzing the data.

Respectfully submitted,

Stephen J. Runde

Stephen J. Runde
Civil Engineer II
Design & Construction

SJR/pc

STABILITY STUDY OF A SLOPE

INTRODUCTION

During construction of I-4, the north half of Vinton Hill Reservoir was drained because of rock blasting in the area and alterations made on the north slope. A retaining wall was constructed adjacent to the north side of the reservoir with an 18" underdrain installed at the foot of the wall to remove any groundwater. However, in the summer of 1971, when the reservoir was back in service, we were informed by the State Highway Commission that there was a continual seepage of water at the eastern part of the north slope. During the following year a slight slippage occurred on this slope due to the supersaturated condition. To try to alleviate this problem a 4" French Drain was installed between the existing pile core and the reservoir wall footing. (See Water Division E & C Drawing No. M515, File M1.) This report deals with a study made to determine the effectiveness of this drain and consequently the stability of the north slope.

MEANS OF ANALYSIS

Three different techniques were used to analyze the slope. First, an arrangement of seven piezometers was installed to locate the groundwater level. These readings reflect the effectiveness of the 4" French Drain in intercepting the water before it flows down the slope. The second step was to collect samples of the water flowing from the 4" and 18" drains. A sample was also obtained from the reservoir. These samples were then tested for total solids and the results of the samples from the drains were compared to the reservoir sample. This was done to check the efficiency of the filtering system around the drains. Using the results of these tests along with the flow measurements (readings taken to determine the quantity of water flowing through the drains) the amount of solids being washed away can be calculated.

With the data obtained from the above mentioned measurements and tests, the internal conditions and/or stability of the earth slope can be determined.

A comprehensive list of the data obtained from this study is included at the end of this report in Tables 1 and 2.

SOIL CHARACTERISTICS

In order to better understand and explain why the failure pattern exist as it does, a description of the soil characteristics is necessary.

The type of soil was realized during the installation of the piezometers. It was described by Brucker & Associates as being "extremely uniform from the ground surface to bedrock" and that it "consisted of a low to medium activity, brown silty clay".

The physical properties of this class of soil were obtained from various references. It is described as having a very high dry strength, however, if not properly compacted it has a tendency to absorb water in which case the soil loses much of its stability. The drainage characteristics are poor to practically impervious depending on compaction and soil mixture.

With these two properties known the analysis of the slide becomes clearer.

ANALYSIS OF PROBLEM

The surfacing of water was "first noticed", as was stated above, in the Summer of 1971. It is safe to assume that the seepage did not occur prior to the construction of 1-4h since the slide failure appeared within one year after refilling the reservoir. That is, the reservoir had been leaking for quite some time as revealed in a leakage report done in 1961, and no seepage was noticed until after construction of 1-4h. The obvious conclusion is that because of the construction, the drainage pattern was altered.

The existing soil has very poor drainage properties and when compacted it is practically impervious. Therefore, water will try to channel its way through inconsistencies in the soil. This statement is supported by the fact that during installation of one of the piezometers water was encountered at depths of 12' and 21'.

The possibility exists that during construction, because of blasting or heavy equipment operations on the slope, the cracks that have developed since the existence of the reservoir were eliminated via collapse or compaction of the soil. Consequently when the reservoir was refilled the leakage had to reroute itself and in this instance channeled its way to the surface.

The solution to this problem would be to provide a "man-made channel" near the source of the leakage. Therefore, the 4" French Drain was installed adjacent to the reservoir.

ANALYSIS OF PRESENT SYSTEM

The 4" drain was installed in the Spring of 1973. This caused the surface of the slope to dry rather quickly which eliminated the surface problem. However, relying purely on the surface condition to say the problem was solved would be unsubstantiated. So, to further explore the subsurface, seven piezometers were installed on the north slope in April, 1975. At the time of installation the north half of the reservoir was again drained because of repairs being made. All piezometer holes were dry during drilling and installation except for #2, in which seepage was noticed. The source must have been either a pocket of water being held by the "impervious" soil or a "channel of water" stemming from the south half of the basin. The seepage found in this hole supports the soil characteristics mentioned before.

On May 15, 1975 approximately five feet of water was raised in the north half of the reservoir for chlorination purposes. The piezometers were read and all were dry except for #2. The reservoir was "full" on the 15th, which was the first

day water was found in some of the other tubes. All piezometers were monitored periodically and the data obtained is shown in Table 2. The water levels fluctuated but seemed to remain stable with no large changes.

To further analyze the data obtained the contour of the bedrock will be outlined. A plan of the approximate location of the piezometer tubes and their elevations is included with this report in later Division Drawing No. 4517. All bottom elevations are at bedrock except for #3.

Using this data, the drainage pattern may be realized. Comparing rock elevations shown and starting at a point around tube #7 it can be seen that water would flow towards the northeast and at some point between the two piezometer rows change direction and flow southeast. It must be kept in mind, however, that this will only hold true once the water has reached the bedrock and be permitted to gradually seep thru the silty clay soil. Channelization may still exist above the actual groundwater level which is what appears to be happening around piezometer #2.

The data obtained partially supports the drainage pattern mentioned. Water was found in tube #7 immediately after the reservoir was full. It only appeared in tube #6 after the water level in #7 was at its peak for about a week. When the level declined in #7 it declined in #6 until #6 was totally dry. Adding to this, the fact that tube #4 contained water only after tube #6 had water, the only possible conclusion is that the water is flowing in the northeasterly direction. The direction of flow at the eastern part of the reservoir cannot be arrived at with the available data. Of more concern here, however, is the elimination of the groundwater. From the data and the drawing on which the "actual" groundwater level is plotted, it can be seen that the 4" French Drain is performing well. (The term "actual" was used because it was assumed that a "channel of water" flows through tube #2, so the readings obtained were not true

groundwater levels, and therefore, this data was not used.)

The 1" drain is intercepting "all" water which seeps over the middle core and the groundwater is probably seeping through the core itself. Since this soil has a very high dry strength and the 1" drain is keeping it dry, the slope is very stable from this standpoint. However, the question still remains as to the efficiency of filters around the French drains. If the drains are washing away the slope it cannot be considered stable, so the total solids tests were run to determine their filtering ability.

The samples obtained from both the 1" and the 1 1/2" drains contained "fine" suspended solids, as could readily be seen by simple inspection. Therefore, the total solids are actually purely dissolved solids.

Because of the daily fluctuation of the total solids in the water pumped to the reservoir, samples were taken every day for about one month to try and develop a pattern. However, after all the data was tabulated a clear pattern could still not be determined. There seems to be no reason for the increase or decrease in the total solids content of the samples taken from the drains on a day to day basis. On the other hand, the data does point out that as the reservoir elevation increases or decreases the flow thru the drains does the same.

Being unable to determine why the daily fluctuation of total solids is so erratic by the above method, it was decided to take a sample from each of the drains and from the reservoir and have them analyzed as to the chemical composition of the solids. The results of the analysis are discussed in the following section.

CONCLUSIONS AND RECOMMENDATIONS

Although it is not possible to look at the data obtained and arrive at a definite conclusion concerning the stability of the slope, a few facts and relationships can be obtained from the data, when they are compiled make it possible

to see the subsurface section.

Enumerated below are the data and conclusions arrived at:

1.) The 4" and 18" drains on the slope are. The 4" drain is catching most of the water trapped above the middle layer and the reservoir and the 18" drain is receiving the remaining water which is leaking thru the middle layer. As the elevation of the reservoir increases the flow thru the drains and the water level in the reservoir increases in head. However, the groundwater elevation never exceeds the elevation of 18" drain, which is a rock pile at the top. In conclusion the slope is remaining dry and because of the characteristics of the soil in the dry state there is a chance of some failure. That is, the slope will not fail by sliding.

2.) Although a sliding failure will not occur, failure may be caused by undermining or piping of the soil. In order to determine whether this is happening a sample was taken from each of the drains, as stated before, and it was analyzed as to its chemical composition.

The results of the tests showed that the increase in total solids was due to an increase in NaCl and HCl, except for a few ppm. These compounds are soluble in water, however, they are not part of the soil structure. The leaching of these does not effect the strength of the soil.

The salts could be deposited in the soil from a couple of sources. One could be due to the winterization of 1-44. The second source was realized after talking to the Parks Department. They said that rock salt is commonly used as a weed killer along fences and walls.

In any case, it is known that neither of these two compounds are part of the brown silty clay soil structure. Therefore, piping of the soil is

not occurring, in the above mentioned cases.

From the data for the 1950-51 season it is concluded that the water level of the reservoir is stable and that the hydrological and meteorological characteristics remain the same. In view of the fact that the data for the 1950-51 season would be without foundation. However, the following data are given for the situation.

The past history of the project in Nevada indicates that it has been at least 1901, at about the same rate as it has now. It is estimated that the time of construction of I-80 was about 1901, and that it was built by the same type of construction of I-80. The evidence also indicates that there was significant damage to the slope. The same situation exists today, and the water is drained by man-made materials. In fact, this water is not filtered before it is filtered before it drains away.

It is safe to assume that a sudden or gradual change in the water level will not occur because of the filtering nature of the filter. A sudden change in conditions would be caused only by a large increase in the leakage from the reservoir. By a large increase it is meant failure of the reservoir structure. This, however, is unlikely to occur because of the stability of the slope.

After analyzing the data obtained in this investigation and comparing it with the past history of the reservoir it is my opinion that further investigation is not needed at this time. Monitoring of the system by taking altimeter readings, flow measurements, and running total gages set on the water level, would be a beneficial check on the effectiveness of the system. Reading taken every three or four months would be frequent enough to detect any changing channels.

In conclusion the obvious solution to the problem would be to halt all leakage from the reservoir. The feasibility of doing this however, is another matter.

REFERENCES

- 1.) Peck, Hanson, and Thornburn. Foundation Engineering, New York, John Wiley & Sons, Inc., June, 1967.
- 2.) Handbook of Chemistry and Physics, Forty-fifth Edition, The Chemical Rubber Co., 1964-65.
- 3.) Lange, Norbert Adolph. Handbook of Chemistry, Handbook Publishers, Inc., 1952.
- 4.) Soils Manual, MS-10, Second Edition, Third Printing, February, 1969.
- 5.) Kammerer, Joseph J. Findings at Compton Hill Reservoir, St. Louis Water Division, 1973.

TABLE 1

RESULTS OF FLOW AND TOTAL SOLIDS TEST

NOTES: Total Solids Test Results From:
Chain Of Rocks Lab. Formula For
Converting To Pounds Per Day:
Pounds/Day = (Difference in Total Solids) x
(8.345 x 10⁻⁶) x (Flow (gal/day)).

Date	Time	Total Solids (PPM)			Flow (gal/day)		Difference #/day (+) Deposited; (-) Displaced		
		h ^m	1 ^m	Res.	h ^m	1 ^m	h ^m	1 ^m	Total
7/5									
7/30	9:00	243	305	243	28,174	12,397	0	-5.38	- 5.38
7/3	9:30	249	301	242	30,422	9,333	- 1.78	-4.60	- 6.38
7/7	1:30	224	305	243	22,189	11,059	+ 7.06	-4.80	+ 2.26
7/10	1:00	231	298	276	27,931	9,310	+10.49	-1.71	+ 8.78
7/14	10:30	254	292	304	27,227	9,801	+11.26	+1.23	+12.59
7/17	11:00	272	339	303	22,192	8,313	+ 5.74	-2.65	+ 3.09
7/24	2:30	293	321	346	30,140	9,698	+12.07	+2.02	+14.09
7/28	2:30	319	345	373	12,756	7,621	+ 5.75	+1.78	+ 7.53
7/31	2:30	364	353	375	17,900	8,549	+ 3.29	+0.78	+ 4.07
7/4	11:45	343	375	370	26,235	8,349	+ 5.91	-0.39	+ 5.52
7/7	10:45	353	366	355	23,060	7,506	+ 2.31	-1.38	+ 0.93
7/11	2:45	353	375	363	20,637	8,952	+ 1.72	-0.90	+ 0.82
7/8	10:45	316	343	334	20,442	8,388	+ 3.07	-0.65	+ 2.42
7/13	2:30	286	316	290	19,938	8,787	- 5.92	-5.40	-11.32
7/25	11:00	317	331	280	19,418	8,844	-19.91	-3.50	-23.41
7/2	9:30	300	325	286	20,280	6,527	- 2.37	-2.14	- 4.51
7/8	4:00	310	353	308	16,322	7,539	- 0.27	-2.35	- 3.12
7/16	10:45	300	333	279	18,402	6,903	- 3.23	-3.15	- 6.38
7/22	11:00	279	308	286	17,143	8,046	+ 1.00	-1.29	- 0.29
7/29	2:15	267	316	287	14,562	8,254	- 1.34	-2.63	- 3.97
7/5	10:30	300	343	325	18,837	6,182	+ 3.93	-0.93	+ 3.00
7/21	2:45	316	333	292	13,500	7,203	- 2.64	-2.15	- 5.13
7/4	2:30	316	333	315	14,368	12,129	0	-1.83	- 1.83
7/11	2:00	292	300	292	16,701	8,222	0	-0.54	- 0.54
7/17	10:00	300	300	290	14,862	9,429	- 6.00	-3.96	-10.26
7/23	1:45	315	324	312	15,244	9,272	- 0.40	-0.23	- 1.33
7/6									
7/22	2:45	267	292	279	12,706	8,861	+ 1.22	-0.65	+ 0.57
7/18	3:00	276	333	276	18,409	10,204	0	-4.9	- 4.9
7/28	1:45	231	240	241	16,119	10,769	+ 4.0	+1.2	+ 5.2
7/8	4:00	325	285	343	13,038	10,608	- 2.0	+4.1	+ 3.1
7/16	3:45	273	300	273	19,877	12,342	0	-2.8	- 2.8
7/20	1:45	364	420	348	16,364	8,752	- 0.13	-4.26	- 7.44
7/3	10:45	250	279	340	17,008	12,313	+12.77	+6.27	+19.04
7/10	10:45	333	400	343	18,621	10,834	+ 2.2	-4.7	- 2.5
7/30	2:00	223	267	285	17,753	9,135	+ 2.3	+1.4	+10.7
7/7	2:15	235	325	273	16,701	9,013	- 1.8	-3.9	- 5.7
7/4	9:30	361	333	342	17,861	10,901	- 2.5	-3.5	- 6.3
7/10	8:50	315	315	343	16,263	8,611	+ 2.1	+1.3	+ 3.8
7/16	8:50	343	325	333	18,244	7,770	- 1.5	+0.5	- 1.0
7/23	8:30	322	313	343	17,426	10,132	- 8.00	+2.54	- 5.55
7/31	8:45	375	343	353	15,241	11,101	- 2.9	+0.9	- 2.0

TABLE 1 (cont.)

Date	Time	Total Solids (ppm)			Flow (gal/day)		Difference #/day (+) Deposited; (-) Displac		
		1" 18"	18"	Disc.	1"	18"	1"	18"	Total
9/27	8:30	315	313	325	16,489	7,964	+ 1.32	-1.19	+ 0.1
10/6	11:30	292	325	286	15,429	7,380	- 0.77	-2.6	- 3.3
10/13	9:30	325	333	300	17,817	10,007	- 2.9	-2.8	- 5.7
10/27	10:30	325	308	267	15,766	4,676	- 7.6	-1.6	- 9.2
11/3	2:30	273	315	273	11,289	2,818	0	-3.4	- 3.4
11/9	2:00	315	315	250	12,414	2,448	- 1.6	-0.79	- 2.3
11/16	3:00	325	325	273	13,466	2,474	- 6.1	-3.3	- 9.4
11/23	9:30	292	315	273	14,831	7,191	- 2.4	-2.7	- 5.1
12/1	1:20	285	300	250	11,551	8,000	- 3.4	-3.6	- 7.0
12/16	2:00	334	334	300	17,705	4,227	- 5.02	-2.83	- 7.85
12/22	2:30	307	315	353	12,366	7,572	+ 4.7	+2.4	+ 7.1
12/28	1:30	315	315	267	17,234	10,340	- 7.0	-4.2	-11.2

TABLE 2

GROUND WATER ELEVATIONS ON NORTH SLOPE OF COMBETON RESERVOIR

NOTE: Refer to the Drawing for the
Location of the Piezometers

Date	Time	Piezometer Tubes No.							Remarks
		1	2	3	4	5	6	7	
9/15	10:30	Dry	132.3	Dry	Dry	Dry	Dry	Dry	Reservoir ear
	3:15	"	132.2	"	"	"	"	"	Approx. 5' of
9/16	10:10	"	131.8	"	"	"	"	"	in reservoir
9/17	3:30	"	130.5	"	"	"	"	"	Full reservoir
9/19	8:50	130.2	136.2	"	130.0	"	"	142.9	
9/20	10:20	"	137.4	"	131.7	"	"	145.8	
9/21	1:30	"	137.5	"	132.0	"	"	146.2	
9/22	9:30	"	"	"	"	"	"	146.3	
9/23	9:00	130.3	137.4	"	"	"	"	144.1	
	3:15	130.4	"	"	"	"	"	144.0	
9/27	9:45	130.6	"	"	132.1	"	"	145.8	
9/29	10:00	130.8	139.0	"	"	"	"	145.4	
9/30	9:30	131.5	140.1	"	132.5	"	"	145.5	
10/6	9:15	131.9	"	"	132.6	"	137.1	145.7	
10/9	10:30	132.0	139.5	"	"	"	Dry	145.5	
10/12	10:30	131.9	139.7	"	132.7	"	138.2	144.3	
10/16	1:40	132.1	139.9	"	132.8	"	139.3	144.2	
10/19	10:30	"	139.6	"	"	"	132.5	144.7	
10/24	2:30	131.9	139.2	"	132.6	"	Dry	144.0	
10/30	1:00	"	138.8	"	132.7	"	140.1	144.6	
11/4	10:30	131.4	138.4	"	132.2	"	140.0	144.1	
11/17	11:00	"	137.8	"	"	"	140.2	143.4	
11/24	2:30	"	138.1	"	"	"	140.8	142.7	
11/29	2:30	"	"	"	132.3	"	139.5	141.6	
12/31	2:30	131.4	138.3	"	"	"	139.2	"	
1/4	11:45	131.3	138.5	"	132.2	"	139.9	143.0	
1/7	10:45	131.4	138.3	"	132.3	"	139.8	142.6	
1/11	2:45	131.5	137.9	"	"	"	139.5	141.9	
1/22	11:15	"	137.7	"	"	"	138.4	141.3	
1/30	10:45	"	137.3	"	132.2	"	132.0	142.2	
2/18	9:30	131.6	"	"	131.4	"	Dry	141.7	
2/25	11:00	131.5	137.0	"	"	"	"	141.5	
3/2	9:30	"	136.9	"	"	"	"	141.0	
3/8	4:00	131.6	137.0	"	132.5	"	"	141.3	
3/16	10:45	"	136.8	"	132.4	"	"	141.1	
3/22	11:00	131.5	136.6	"	132.3	"	"	140.4	
3/29	2:15	131.4	"	"	132.4	"	"	140.5	
4/5	10:30	"	136.5	"	132.3	"	"	141.1	
4/20	2:45	131.5	136.3	"	132.4	"	"	141.3	
4/24	2:30	"	136.5	"	132.5	"	"	141.5	
4/31	2:00	131.2	136.2	"	132.1	"	"	141.1	
5/17	10:00	"	136.1	"	"	"	"		Res. lock frozen
5/23	1:45	131.1	135.7	"	131.9	"	"	141.0	couldn't get h 2

TABLE 2 (cont.)

Date	Time	Pierometer Tubes No.						Remarks
		1	2	3	4	5	6	7
7/76								
7/7	10:30	131.0	135.5	Dry		Dry	Dry	Res. lock fro.
7/22	2:45	130.5	134.4	"	131.6	"	"	couldn't get
7/29	10:30	130.4	"	"	"	"	"	"
7/11	2:15	"	135.1	"	"	"	"	140.6
7/20	2:00	"	134.0	"	"	"	"	"
7/4	3:30	"	135.4	"	131.7	"	"	141.1
7/11	4:00	130.6	135.5	"	131.8	"	"	143.0
7/18	2:30	"	"	"	"	"	"	"
7/25	4:30	"	135.4	"	131.7	"	"	143.1
7/1	4:15	"	135.1	"	131.6	"	"	"
7/8	4:00	130.5	134.4	"	"	"	"	142.1
7/14	4:00	"	"	"	131.4	"	"	"
7/22	3:15	"	134.8	"	"	"	"	"
7/29	3:30	130.4	134.5	"	131.6	"	"	140.6
7/6	1:45	"	134.4	"	131.5	"	"	140.8
7/20	1:45	130.3	134.1	"	"	"	"	140.3
7/3	10:45	130.1	134.2	"	131.6	"	"	142.0
7/10	10:45	130.3	134.3	"	"	"	"	141.3
7/30	2:00	130.4	134.8	"	"	"	"	140.8
7/7	2:15	"	"	"	"	"	"	140.3
7/19	2:45	130.2	133.5	"	"	"	"	140.8
7/27	9:45	134.2	132.2	"	131.2	"	"	140.1
7/2	9:30	132.4	133.2	"	131.4	"	"	141.1
7/10	8:50	130.4	133.8	"	"	"	"	139.9
7/16	8:50	132.1	134.3	"	131.6	"	"	140.4
7/23	8:30	130.5	134.0	"	131.5	"	"	139.8
7/31	8:45	130.2	133.5	"	"	"	"	"
7/14	2:45	130.1	"	"	"	"	"	139.9
7/27	8:30	129.5	132.7	"	"	"	"	139.8
7/6	11:30	134.1	133.3	"	"	"	"	140.8
7/13	9:30	130.8	"	"	"	"	"	139.8
7/20	11:00	130.2	132.7	"	131.6	"	"	140.1
7/27	10:30	131.8	132.8	"	131.5	"	"	140.4
7/3	2:30	"	132.9	"	"	"	"	140.7
7/9	2:00	130.8	132.8	"	131.4	"	"	140.1
7/15	3:00	130.4	132.6	"	"	"	"	"
7/23	9:30	130.2	132.5	"	131.3	"	"	140.3
7/1	1:30	130.4	132.6	"	131.5	"	"	140.2
7/9	2:30	132.3	"	"	"	"	"	139.8
7/16	2:00	130.8	"	"	"	"	"	139.7
7/22	2:30	130.2	132.3	"	131.3	"	"	139.2
7/28	1:30	"	132.0	"	131.4	"	"	139.8
7/77								
7/15	8:30	137.6	134.0	"	132.0	138.1	140.1	143.8
7/31	9:00	137.3	134.2	135.5	"	137.2	139.5	144.2
7/11	1:45	132.2	132.6	Dry	"	136.3	137.6	143.2
7/21	10:30	131.2	132.2	"	"	136.5	137.1	141.9
7/29	2:45	130.8	132.0	"	131.3	136.7	138.1	"

Compton Hill Reservoir - French Drain Flow Readings

Date	Time	4" Drain			18" Drain			
		#1	#2	#3	#1	#2	#3	
5-8-78	10:30 AM	18.8 sec.	18.8	19.1	7.0	7.6	8.2	FW
5-8-78	8:45 AM				LOWER	DRAIN VERY DIRTY		
5-15-78	8:45 AM	25.6 sec.	24.7	23.8	9.5	10.0	9.5	FW
6/7/79	2:00 PM	10.3	9.6	10.9	5.1	7.1	5.0	
6/5/79	8:30 AM	13.4	13.6	13.0	6.5	5.5	5.5	LRM, J.S.
6/3/79	1:50 P.M.	16.1	16.5	16.3	5.8	6.6	5.8	LRM, J.S.
7/4/79	8:30 a.m.	13.9	13.8	13.8	6.2	6.3	6.2	FW, RS
12/9/79	9:00 AM	15.8	15.9	16.2	5.5	5.5	5.2	FW, RS

Compton Hill Reservoir - French Drain Flow Readings

Date	Time	4" Drain			18" Drain			Crew
		#1	#2	#3	#1	#2	#3	
2-3-79	11:00 AM	21.5	21.2	21.2	6.3	6.0	6.2	FW, J.S., RS
7-7-79	1:00 PM	23.1	23.4	22.8	5.8	5.5	5.4	RS, J.S.
9-2-79	1:30	20.1	19.3	19.4	5.6	5.4	5.5	
11-11-79	9:00 AM	20.5	21.4	20.8	6.1	5.8	5.7	RS, J.S.
1-3-80	9:50 AM	16.1	17.0	16.5	5.2	5.6	4.3	FW, LRM, J.S.
1-23/80	10:00 AM	16.7	16.3	16.8	5.3	5.6	5.2	FW, J.S.
1-26/81	2:15 PM	17.3	17.3	17.2	5.5	5.5	5.5	T.B., B.F., FW
1-6/81	9:20 AM	18.9	18.4	18.6	7.1	5.8	5.5	FW, T.B., B.F.

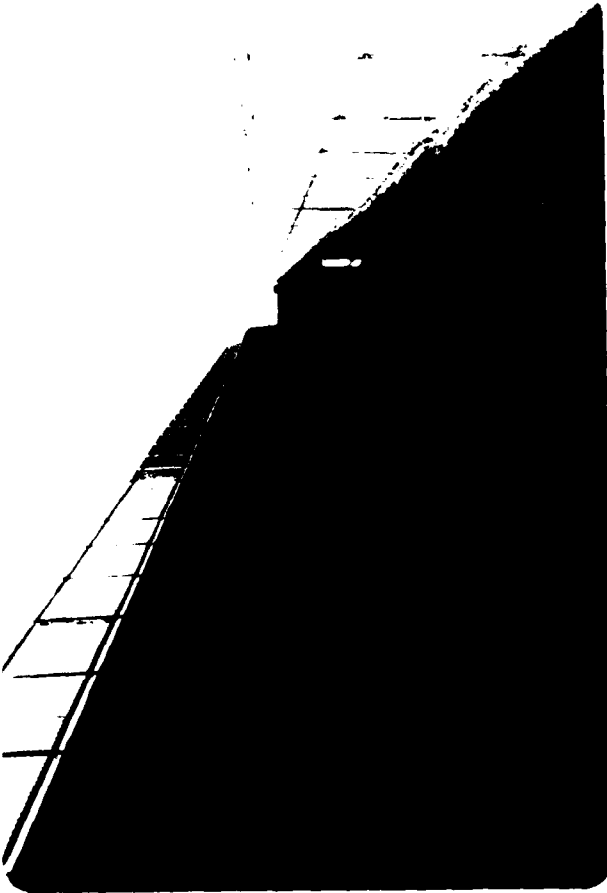
Compton Hill Reservoir Ground Water (Piezometer) Elevations																	
Date	Time	Tube #1		Tube #2		Tube #3		Tube #4		Tube #5		Tube #6		Tube #7	Remarks	K.A. No.	-DC Black
		Top of Tube	Water	Top of Tube	Water	Top of Tube	Water	Top of Tube	Water	Top of Tube	Water	Top of Tube	Water				
7-2-79	11:00 AM	28.2		32.3		DRY		50.0		GONE		GONE		38.6			
8-3-79	11:30 AM	28.3		33.1		Dry		49.9		-		-		38.7			
8-2-79	1:30 PM	28.4		33.2		Dry		50.1		-		-		38.7			
8-2-79		28.5		33.3		Dry		50.2		-		-		-			Plugged #7
8-2-79	9:30 AM	28.8		33.3		Dry		50.1		-		-		38.7			
8-23-79	10:30 AM	28.1		33.2		Dry		DRY		-		-		-			Plugged #7
8-27-79	10:30 AM	28.3		33.3		Dry		DRY		-		-		-			
8-28-79	8:30 AM	28.6		32.6		DRY		DRY		-		-		38.2			

CHECKING IRON RODS ON AND SOUTH C.H. RESERVOIR

DATE	UPPER ROW (SOUTH)				LOWER ROW (NORTH)			
	25'W	50'W	75W	100W	25'W.	50'W	75'W	100'W
1-8-78	INSTALL RODS							
2-5-78	OK	OK	OK	OK	OK	OK	OK	OK
3-6-79	OK	OK	OK	OK	OK	OK	OK	OK
4-5-79	OK	OK	OK	OK	OK	OK	5/8" N.	5/8" N
1-30-79	OK	OK	OK	1/4" N	OK	OK	" "	" "
5-1-79	OK	BENT OVER BY MOWER	BENT OVER BY MOWER	"	OK	OK	" "	" "
6-29-79	7/8" No	BENT	BENT	BENT	OK	OK	BENT	OK OK
8-3-79	7/8" No	Bent over →			OK	OK	Bent	OK
9-2-79	Bent	"	"	"	OK	OK	"	Bent

APPENDIX A

INSPECTION PHOTOGRAPHS



1	2
3	X

PHOTO KEY

<u>NO.</u>	<u>DESCRIPTION</u>
1	North Wall of Reservoir
2	North Embankment - Note Slump Bulge at Stairs
3	Piezometer in Top of North Embankment





4	5
6	X

PHOTO KEY

DESCRIPTION

NO.

4 East Wall of Reservoir

5 East Embankment

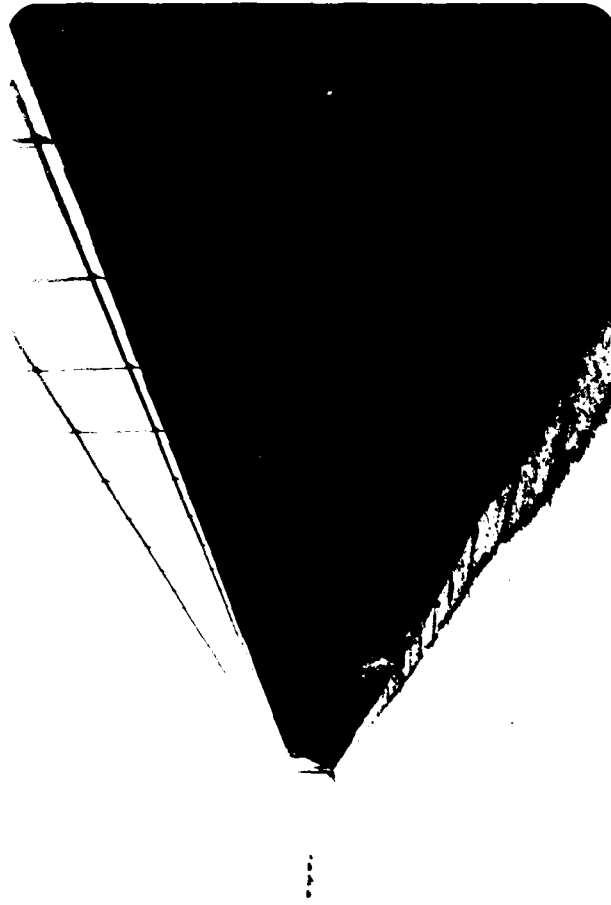
6 Deteriorated Area of East Wall



7	8
9	X

PHOTO KEY

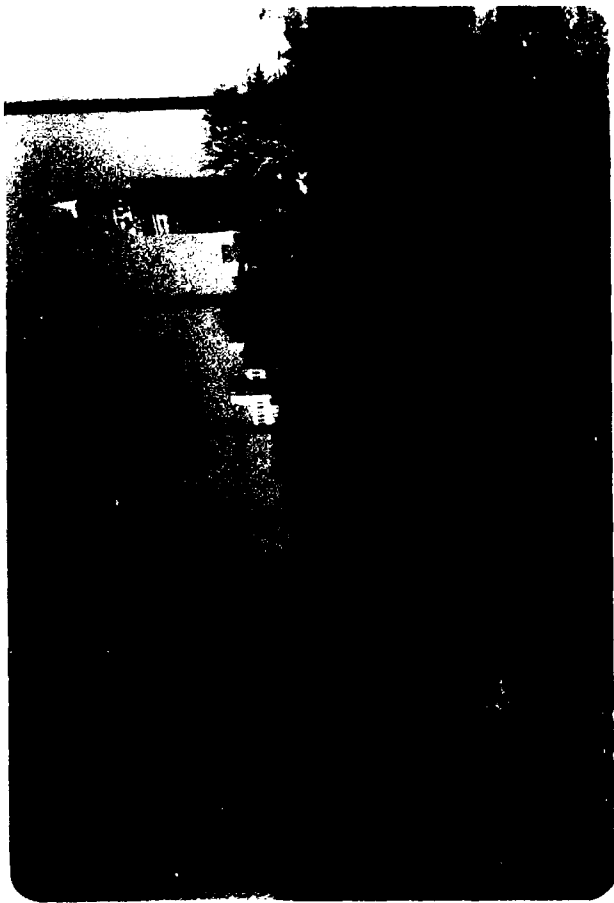
<u>NO.</u>	<u>DESCRIPTION</u>
7	South Wall of Reservoir
8	South Embankment
9	Deteriorated Area of Southwall



10	11
12	X

PHOTO KEY

<u>NO.</u>	<u>DESCRIPTION</u>
10	West Wall of Reservoir
11	West Embankment
12	Deteriorated Area of West Wall



13	14
15	X

PHOTO KEY

DESCRIPTION

NO.

- 13 Roof Above South Basin
- 14 West Gate House
- 15 Last Gate House and Manual Gate Operators

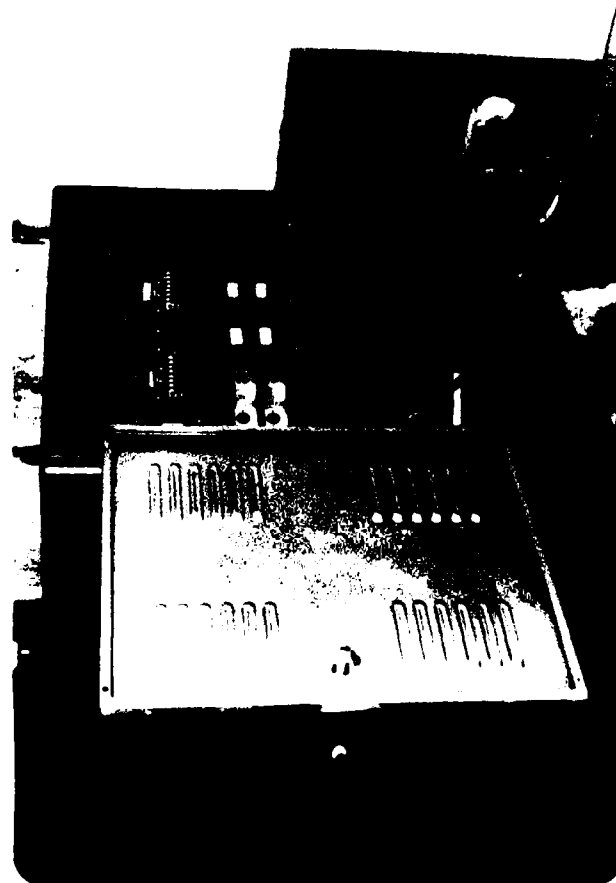
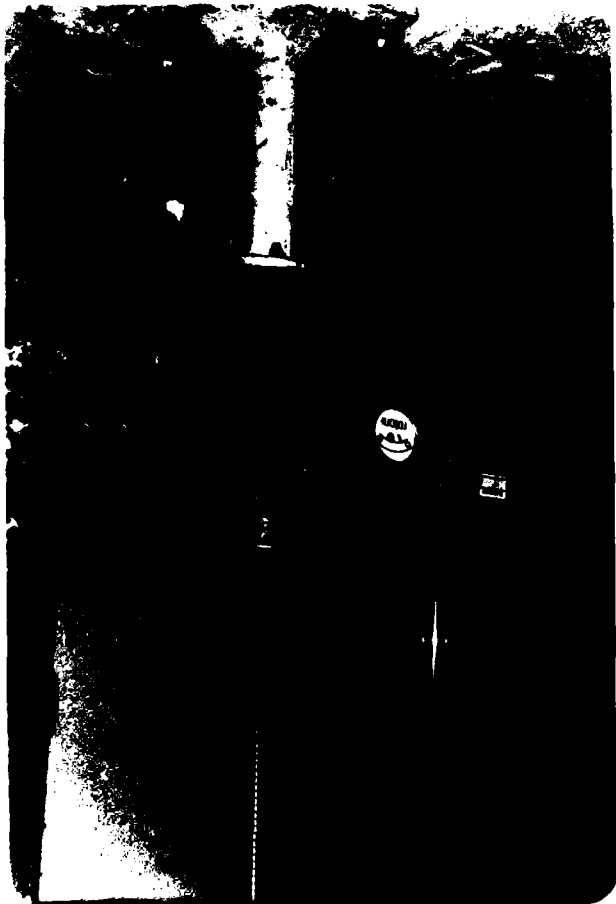
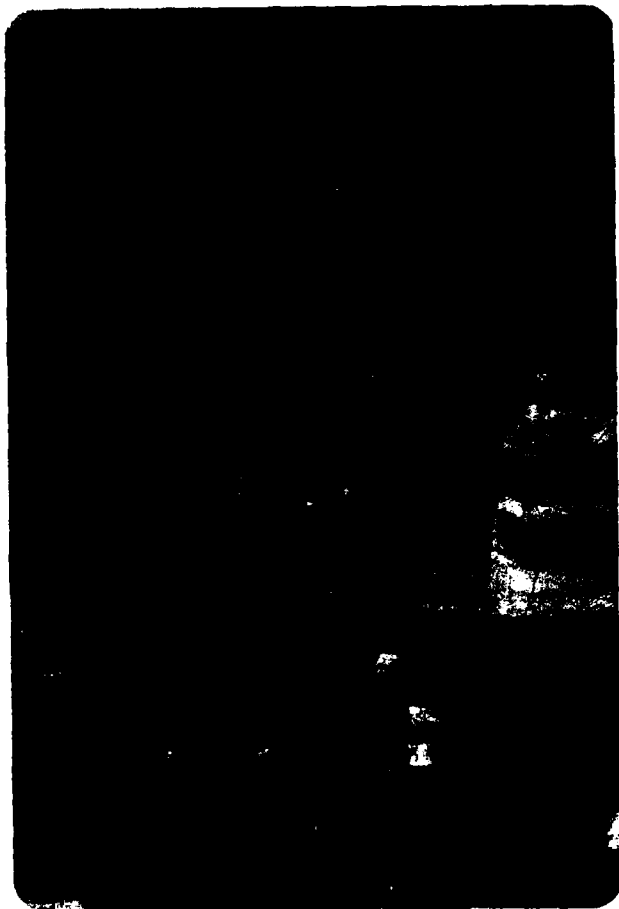
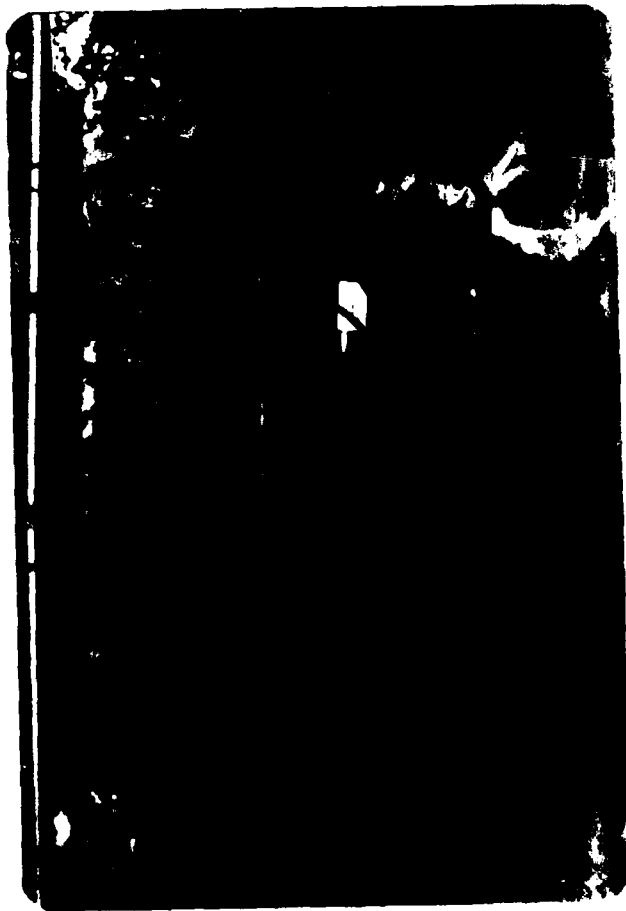


PHOTO KEY

16	17
18	

<u>NO.</u>	<u>DESCRIPTION</u>
16	Motorized Gate Operator at West Gate House
17	Water Level Gage and Manual Gate Operator - At West Gate House
18	Telemeter Controller at West Gate House



19	20
21	X

PHOTO KEY

<u>NO.</u>	<u>DESCRIPTION</u>
19	Emergency Generator at West Gate House
20	Spalling and Lime Deposits at Cracks in Concrete Wall
21	Exposed Rebars in Top of Wall

APPENDIX B

HYDRAULIC ANALYSES

APPENDIX B

HYDRAULIC ANALYSES



HORNER & SHIFRIN, INC.
CONSULTING ENGINEERS
5200 OAKLAND AVE ST LOUIS, MO. 63110

SHEET NO.

OF

JOB NO.

2528

SUBJECT FILE

Res. Drain

TITLE:-

CH Reservoir

BY

ASB

DATE

4-26-81

CHECKED

DATE

INVESTIGATION OF
DRAIN & OVERFLOW CAPACITY

CONSULTING ENGINEERS
5200 OAKLAND AVE. ST LOUIS, MO 63110

SHEET NO

3 6

JOB NO

228

SUBJECT PAGE

Res. Vrain

• **Q**

DATE _____

CHECKED

DATE _____

TITLE:- CH Reservoir

1.3.3

4-20-81

CHECKED

DATE _____

120

172

091

11

33

100 + 100 = 200

142-111717 @ 0.023

I. v. 131.1

2. Многочлен

PROFILE 25 SEWER

Scale: 1:1 Showw

2002-193

Inv. 1249

021

V
T R

$$V_{\text{stat}}^{\text{AA}} + V_{\text{val}} = 57 + \frac{10}{67} = 57.15$$

Note: Same level of approx.

Elavethwin - City Datum

B-2

Country Hill Place
1750 Avenue
Tues. 1928

2761-136.8

Entradas
El. 127.9

Inv. 1249

021



TITLE:

CH Reservoir

SUBJECT FILE

Res. Drain

BY A.B.S.

DATE 4-20-21

CHECKED

DATE

Sewer Capacity

Drain = 20" CI Pipe
Area = 2.18 sf

Assume $n = 0.015$

$V = \frac{Q}{A}$

(1) For $Q = 20$ cfs
 $V = 9.15$ ft/sec.
 $\frac{V^2}{2g} = 1.31'$

$$V = \frac{0.590 d^{2/3} s^{1/2}}{n} \quad (\text{Manning})$$
$$= \frac{(0.590 \times 1.41) s^{1/2}}{0.015}$$
$$= 55.46 s^{1/2}$$

$$\therefore s^{1/2} = \frac{V}{55.46} = \frac{9.15}{55.46} = 0.165$$

$$s = 0.027 \text{ ft.}$$

(2) For $Q = 25$ cfs
 $V = 11.47$ ft/sec. $\frac{V^2}{2g} = 2.04'$
 $s^{1/2} = \frac{11.47}{55.46} = 0.207$
 $s = 0.043 \text{ ft.}$

(3) For $Q = 30$ cfs
 $V = 13.76$ ft/sec. $\frac{V^2}{2g} = 2.94'$
 $s^{1/2} = \frac{13.76}{55.46} = 0.248$
 $s = 0.062$



TITLE:

CH Res.

(4) For $Q = 35 \text{ cfs}$
 $V = 16.05' / \text{sec.}$ $\frac{V^2}{2g} = 4.00'$
 $S^{1/2} = \frac{4.05}{55.46} = 0.290$
 $S = 0.084' / \text{ft.}$

(5) For $Q = 45 \text{ cfs}$
 $V = 12.25' / \text{sec.}$ $\frac{V^2}{2g} = 3.05'$
 $S^{1/2} = \frac{12.25}{55.46} = 0.221$
 $S = 0.107' / \text{ft.}$

Reservoir Inflow —

Max. Pumping Capacity @ Chain of Rocks
 Treatment Plant = 116 MG/D* (40 pumps @ 2.9
 Assume 2 pumps off.
 ∴ Forward Flow, say 40 MG/D.
 Assume 50% flow to Distribution System.
 ∴ Assume res. Inflow = $\frac{0.5 \times 40,000,000}{7.5724 \times 60 \times 60} = \underline{\underline{21 \text{ cfs}}}$

* See 3rd. Inter. Division.

TITLE: CH Res.

Hydraulic Gradient @ Res. Sewer Chamber -

Q Intake H _g (ft.) (60) Grad. NH#1	H _g (ft.) D _g > S	DS Elev. NH#2	H _g (ft.) NH#2	US Elev. NH#2	H _g (ft.) D _g > S	H _g (ft.) Value	DS Elev. S, Cham. S, Cham. S, Cham.	H _g (ft.) H _g (ft.)
20 127.9	8.4	136.3	2.0	138.3 (a)	2.8	0.1	151.1	2.0
25	13.3	141.2	3	144.3 (a)	6.1	0.2	153.5	3.1
30	19.2	147.1	4	151.5 (a)	8.8	0.3	160.6	4.4
35	26.0	152.9	6	159.9 (a)				
40 127.9	33.2	161.7 (b)						

Conclusion: Max capacity of 20" sewer drain = $\frac{30 \text{ cfs}}{2}$

* Assume head loss = $1.0 \frac{L^2}{24}$
 ** Assume " " " " 20 " " " " = $0.10 \frac{L^2}{24}$
 (a) Assume initial h_g gradient to be
 introduced 20" pipe = El. 147.2
 (b) Excess top of 16" hole elevation.



HORNER & SHIFRIN, INC.
CONSULTING ENGINEERS
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SHEET NO.	6	6	JOB NO.	7072	
DESIGNED BY	J. S. Vrain				
BY	ASD	DATE	Aug-28	CHECKED	DATE

TITLE-

CH Kcs.

Investigate Capacity of Kcs. Overflow

(See water furnished)

$$Q = CH^{3/2}$$

Assume $C = 3.1$ if weir is not negligible.

Top openings $H = 1.4' H = 3.0' W$ (each)

Bottom opening $H = 1.4' H = 3.0' W$ (each)

<u>H (ft)</u>	<u>Q (cfs)</u>	<u>S (ft)</u>	<u>h₁ (ft)</u>	<u>h₂ (ft)</u>	<u>Q (cfs)</u>	<u>h₃ (ft)</u>
0.25	4.6	1.45	0.14	126.89		
0.50	13.2	2.10	0.28	127.18		
0.75	24.2	2.61	0.41	127.46		
1.00	37.2	3.10	0.55	127.75		
1.25	52.0	3.47	0.69	128.04	7.60	

11/12 Aug.
1954

Conclusion: Total capacity of overflow openings is 26.19 cfs

DA
FILM